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More New Haven Trouble

EARLY this week the Interstate Commerce Commission made public its report on the financial transactions of the New York, New Haven & Hartford Railroad. While the report is a scathing arraignment of the Mellen management and the directorate for permitting "one of the most glaring instances of maladministration in all history of American railroading," the facts are already familiar to those who have followed the case in the daily press. The property is now in good hands, and this journal believes the Elliott administration can be relied upon to establish order out of chaos. Whether the directors can be made to restore the money squandered, as proposed by the commission, remains to be seen. The Engineering Record is more interested in prevention than in cure. It is lamentable that all this jugglery of millions could have gone on under the noses of commissions, powerless to interfere. The fault is not with the commissions, but rather with their powers and duties as prescribed by law. When proper legislation goes through, as seems inevitable, for the Federal control of the issue of securities, Mellenism should become impossible.

Brightening Trade Outlook

THERE is a rift in the cloud of gloom which has hung over the industrial world for the past year. Reports from widely scattered sections of the country—from industrial leaders and bankers—indicate conditions which should make for larger business in the second half of this year than in the first. Chiefly responsible, of course, are the abundant crops, already reaped and in prospect. In some branches improvement is already evident, as has been apparent in the steel trade. While the new business there is not remarkably large, there is a sufficient upward tendency to take the sign as a hopeful one and not a mere temporary condition. The sky, however, is not entirely clear, and will not be until Congress adjourns. Only then can there be certainty as to the character of legislation which will affect business. Whereas the demand some weeks ago was for adjournment without passing any laws affecting trade, the sentiment now seems to be that haste in passing what must be passed is more important than the exact character of the legislation. This tone may be due, in part, to the apparently changing attitude of the President, which seems to be less uncompromising than it was some weeks ago. If now the Interstate Commerce Commission can reach a decision soon, the waiting period will be over. Engineers have felt the depression keenly because of the curtailment of private construction. They will welcome the turn no less joyfully than those whose investments in industry and transportation have been seriously effected in the last twelvemonth.

Creosoted Piles

WITH reinforced-concrete growing in favor for piles in teredo-infested waters, the highly satisfactory results obtained with creosoted timber on the Pacific Coast, as recorded on page 66, are worthy of careful attention. Probably few waters are more infested with marine borers than those of California. Yet many engineers, notably those of the Southern Pacific Company, cling to the treated timber pile and get satisfactory service from it, even using Douglas fir, which, untreated, is extremely shortlived. It goes without saying that the pile must be carefuly handled before it is driven to position. If the surface of the timber is injured by hooks, spikes or other causes so as to give the teredo an easy entrance, the stick is doomed. Taking into consideration the costs as given, however, it is evident that with proper care in seasoning, treating and driving the creosoted pile amply justifies itself.

Acoustics of Auditoriums

MUCH experimental work has been done to determine the best methods for correcting acoustical defects of auditoriums. The work of Professor Sabine in this line has become classical. Few people, however, understand to what extent experiments have been carried to remedy the defects of a given room. For that reason a bulletin just issued by the Engineering Experiment Station of the University of Illinois will probably find a wide circle of readers. For about six years studies on the auditorium of that institution have been under way under the direction of Dr. F. R. Watson. The acoustical properties of the room were wretched at the start, but by a series of experiments within the auditorium itself, following careful studies in the laboratory, the trouble has been, practically, cured. Echoes were located by means of an alternating current arc light, a beam from which, accompanied by the hissing sound of the arc, was directed to various parts of the room. The paths of the light reflections were traced and verified by the sound. A complete acoustical survey was thus made of the auditorium, resulting finally in the hanging of curtains and canvases in various parts of the room so as to stop the reverberations. Such experimental work is of the utmost importance to those engaged in the design of buildings for accommodating public gatherings, because the analysis of the trouble leads to the formulation of principles for their avoidance. The pamphlet is therefore an important addition to the literature of practical acoustics.

International Specifications

IT will be remembered that at the New York Congress of the International Association for Testing Materials the outlook for progress in writing international specifications was not considered hopeful. The congress merely directed further study of the question. Progress on the specifications for steel and cast iron, however, has been more rapid than was expected. Not only have the committees continued their studies but at the Turin meeting of the council of the association in April of this year international specifications for both steel and cast iron were presented. While taking no final action on them, the council gave evidence of its approval of the progress and directed that the activities of the committees be continued. Moreover, that its attitude might be thoroughly understood a resolution was introduced declaring the council's approval of their preparation. It asked that there be full discussion and trial of the standards so far presented, so that if possible final action may be taken at the St. Petersburg congress in August, 1915. The resolution had the hearty support of influential members present and was passed without opposition. While the difficulties in the way are great the progress in the last two years has been decidedly encouraging. Certainly American engineers will agree with the council in its statement that it sees "in the agreement upon standard specifications for export purposes a distinct advance in international efficiency and economy."

Promoting Export Trade

THE stabilizing influence of a large, diversified and well distributed export business upon home conditions has long been recognized. Any movement, therefore, for aggressively and systematically extending our export trade is of interest not only to the manufacturers who are seeking wider markets but to the country at large. A greater foreign trade spells greater prosperity and less likelihood of complete stoppage of business when domestic conditions are unsatisfactory. Pursuant to a resolution adopted at the National Foreign Trade Convention in Washington last May, a Foreign Trade Council composed of thirty-four executives, representing various branches of industry, transportation and finance, with Mr. Farrell, president of the United States Steel

Corporation as its head, has been organized. The object of the council is to secure the active co-operation of all elements concerned in the development of an oversea commerce, in contradiction to the policy heretofore obtaining, of individual initiative in studying and developing foreign markets. The intent was plainly shown in a resolution adopted at the convention, declaring that "the development of the United States makes it essential to the best interests of the Nation that the Government and the industrial, commercial, transportation and financial interests should co-operate in an endeavor to extend our foreign trade." The new council will meet early in the fall and will at once establish co-operation with the Chamber of Commerce of the United States. The character of the membership is a guarantee that the work will be aggressive and effective. If the movement denotes the beginning here of an alliance in foreign trade promotion such as exists between German manufacturers and their Government important extensions in all lines of foreign commerce may confidently be looked for in the near future.

Machinery Back of the Rivers and Harbors Bill

COMMENTING upon the Rivers and Harbors Bill now before Congress General Kingman, Chief of Engineers, has given out a statement protesting against the condemnation of the projects included in the measure. He points out that all of them have been studied by the Corps of Engineers—under a local or district engineer—have been modified and approved by the division engineer, have been passed upon favorably by the Board of Engineers for Rivers and Harbors and, finally, by the Chief of Engineers, before being transmitted to the Secretary of War and thence to Congress.

It need hardly be said that criticism of the measure now before Congress, or of other rivers and and harbors bills, is not necessarily a criticism of the Corps of Engineers, for their work consists merely in the study of each project by itself, without reference to its merit as compared with other projects. The difficulty comes in putting these various projects together into a bill, for which Congress alone, or possibly the constituents behind Congress, is responsible. Recognizing the ability of the Corps of Engineers it is a safe assumption that if it were entrusted with the responsibility of determining the country's policy in river and harbor improvements and with the drafting of the necessary legislation, some plan more conformable to business procedure would result. This is said in full recognition, too, of the difficulties involved. While unquestionably senators and representatives do trade about a great deal in the formulation of a bill, there is insistent demand from all sections of the country for improvement of streams and harbors. Even the Corps of Engineers, if entrusted with complete control of the measure before presentation to the legislative body, would find it no little task to convince disappointed delegations from various sections of the country of the superior merit of the plans approved.

The progress toward a better policy must largely be one of education, and it is only through an active campaign, such as Senator Burton is conducting, on proposed measures that correct ideas will be disseminated through the public prints, reach the people and thus influence their congressional representatives.

Senator Burton's efforts are directed toward securing a definite and economically sound policy in river and harbor appropriations. To this end he recommends, among other things, that provision be made for the completion of a project in the bill in which it is adopted. Of hardly less importance is a policy of improving the main stream before giving attention to the branches, and of determining depths and dimensions with a view to a comprehensive and uniform plan for the development of such waterways as can profitably be utilized. In a word he seeks a policy that will result in a weighing of all projects proposed and in their adoption in accordance with some logical sound plan. A bill founded on such a policy would be a co-ordinated measure and not a mere patchwork, as have been most of the rivers and harbors bills of the past.

New York's Mammoth Pier

THE improvements now under construction in the North River, New York, by the Department of Docks will eventually provide a 150-ft. x 1050-ft. pier at West Forty-sixth Street and a corresponding half pier at West Forty-fourth Street with adjacent slips 360 ft. wide and 44 ft. deep giving accommodations for the simultaneous berthing of three of the largest steamships now entering or likely to enter the port. The total estimated cost of the construction, although not yet officially announced, will be very large as indicated by a sum of $500,000 for the preliminary work described on page 68 of this issue. The construction will be unusual in that a portion of the large pier is supported directly on solid rock and the remainder on piles carrying heavy live and deadloads through friction alone without any possibility of their reaching to solid bearing in hard material. Full-size tests of clusters of piles indicate that friction enough can be developed to sustain loads properly distributed over the available area. Nevertheless, this radical difference in adjacent portions of such an important structure will be observed with interest equal to that in the successful construction of the new Municipal Building, with its enormous steel superstructure supported partly on quicksand and partly on rock.

The central metropolitan location of these piers, so quickly and easily accessible from the great passenger terminals and hotels of the city, will undoubtedly be a convenience to passengers and an advantage to the steamship lines which is more pronounced because so much superior to that of most other great seaports. It is, however, to be seriously questioned whether these advantages, which are certainly not vital ones, are commensurate with the great cost of the undertaking which will materially add to the city's enormous liabilities and interest burden. There are other objections—the obstruction of the public highway of the river, none too wide at the best; encroachment upon the valuable space at the land end; and finally the positive increment which will be added to the congestion of city traffic rather than the relief that might be obtained if the piers were located on Long Island or on Staten Island where they could be constructed much more cheaply, quickly and advantageously, could have unlimited space, and would still be thoroughly convenient and much nearer, both in point of time and distance, than are the steamship piers of many other large cities to their civic centers.

A broad gage policy for New York that would classify railroad, steamship, industrial and warehouse interests and establish them with ample provision for perfection and future extension in districts where they could be advantageously constructed and properly connected with the center of the city would be far more economical and result in much greater progress, prosperity, convenience and beauty for the city than the unbalanced crowding of new constructions adding new business to the intensely congested area of Manhattan Island.

Letter Ballot under Fire

ONE of the liveliest discussions at the recent meeting of the American Society for Testing Materials was that on the present procedure in adopting specifications. It was brought about by Professor Marburg's paper reviewing the present methods. In the first place, question was raised as to the competence of the annual meeting to pass on, either amending or accepting, the specifications as prepared by the technical committees. The second objection was against the letter ballot which, experience has shown, to be practically valueless.

While there is much truth in the first objection—that a heterogeneous meeting is not competent to weigh a highly technical specification—the society's history shows that the authority vested in the general meeting has frequently been exercised to the benefit of the organization. While it is true that those present, with the exception of the specialists on the material under consideration, cannot qualify as experts, nevertheless appeals to their judgment made by written communication or in person, answered on the floor by the chairman of the committee, have been intelligently answered. At the last meeting some reports were referred back to the committee on account of objections made, while still others, objected to just as strongly, were passed to letter ballot, because the testimony of the chairman showed that the objections had been fully considered and rejected by a majority of the members of the committee. It would seem, therefore, that while some change might be advisable in the percentage vote for passing or amending specifications, the power of the annual meeting has been exercised in a way that makes it advisable that it be retained.

As to the strictures on the letter ballot there was quite general approval. Never has a specification, passed by the annual meeting, been rejected by letter ballot, and, as a rule, only 6 or 10 per cent of the members vote. On the specifications submitted to letter ballot last year there were only ninety-five votes from a total of 1620 members, the average affirmative and negative votes being ninety-two and three, respectively. This showing is not surprising, for under present procedure each member has been apprised in advance of the meeting of proposed specifications and has had an opportunity to present his objections thereto either personally or in writing. Failing to carry his point in the meeting, if objection be made, there is little likelihood of success in the letter ballot. Consequently, there is apathy on the part of both the opponents and the proponents of a given standard.

In support of the leter ballot plan, it might be pointed out that standards which have been amended by the annual meeting would fail to have the scrutiny of the entire organization if the letter ballot were abolished. In other cases there seems no reason for its continuance. Here again moreover, the argument applies that the society as a whole is incompetent to pass upon a highly technical specification. It would seem, therefore, that if the letter ballot is to be retained it should be modified so that an especial appeal is made to the experts as to each class of specifications. If they fail to object the specifications might be considered as adopted, while if a certain number of them, say fifteen, disapprove of the specification either in whole or in part, such disapproval would act automatically to send the standard back to the committee for consideration, despite favorable action thereon by the annual meeting.

With some such procedure—special appeal to the experts in each line, and an opportunity to suspend the adoption of specifications by the objections of a reasonable number of members—what virtue may lie in the letter ballot would be retained. There is little doubt that the interest aroused by Professor Marburg's paper will lead within a reasonable time either to its abandonment or modification along more effective lines.

Pulverized Coal for Steam Boilers

MORE than a score of patents for pulverized coal apparatus have been issued in the United States during the past twenty years, and attempts to utilize this low-grade fuel in steam generation are constantly being made in commercial installations. According to a paper recently presented by F. R. Low before the American Society of Mechanical Engineers, smokeless combustion and high efficiency appear to have been attained. However, maintenance and the securing of full boiler capacity still offer serious difficulty.

The proportionately large surface exposed for oxidation by the finely comminuted fuel makes perfect smokeless combustion with a minimum air supply possible, but generally with excessive furnace temperatures. If these are reduced by increasing the excess air, efficiency is sacrificed; if by reducing the amount of fuel supplied, the capacity is limited. The cost of cleaning boilers and the expense of renewing burned-out furnace linings, combined with the fact that most grades of coal can be burned with a reasonable degree of smokelessness in the cheaper stoker-equipped plants in common use, and at an efficiency closely comparable to the best results with pulverized fuel, militate against the general application of the latter at present.

Pulverized coal can be safely handled if care is taken that it shall not be blown or sifted about in a finely disseminated state, and if the flow of the mixture of coal dust and air exceeds the rate of flame propagation in systems where the pipe back of the blower is filled with an explosive mixture. In later forms of apparatus considerable flexibility of adjustment of air pressure and fuel ingress is provided; preheating of air tends to increase the operating efficiency, and the pulverizing, separating and blowing equipment is carefully designed.

In localities where high grade fuel is at a premium, but where there is an ample supply of poorer quality, the application of equipment suited to burning pulverized coal is likely to receive an increasing amount of attention in the near future.

Small Prime Movers

THERE have been published many studies of the efficiency and operative qualities of such small sources of power as are convenient for many purposes. Few such investigations, however, have covered any considerable range as regards the circumstances of use and the type of machine studied. A recent paper by Dr. Straus in the "Electrotechnische Zeitschrift" gives perhaps the most thorough and up-to-date discussion of the comparative costs of small prime movers that has yet been published, and its results deserve wide attention. The particular purpose of Dr. Straus' study was to compare the cost of power generated from small prime movers with that attainable by electric motors using central station service.

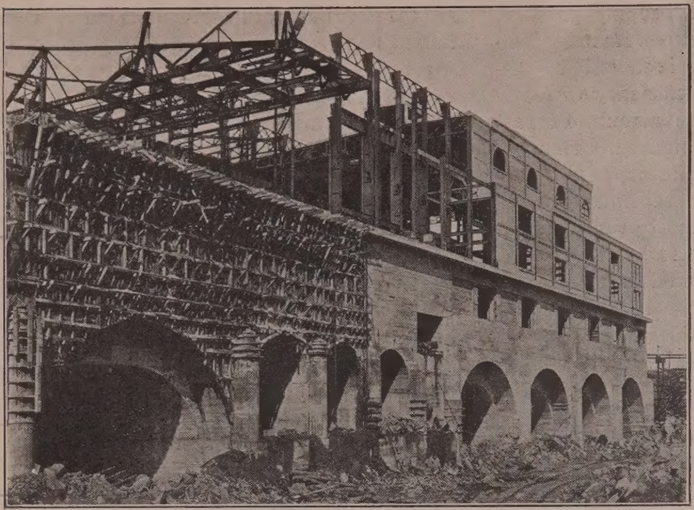
The typical small machine selected was one of 20 actual horsepower corresponding to cases frequently found in the installation of temporary power equipment for small work. The prime movers investigated were the ordinary portable steam engine, locomobile (that is, the little self-contained high efficiency steam units common in Continental practice), gas engine with ordinary illuminating gas, producer gas engine, gasoline engine, benzol engine, and finally the Diesel motor using comparatively heavy oil. Full account was taken in the investigation of the cost and up-keep of the various units as well as their efficiency at various loads and for various hours of use per year.

It is quite difficult in making the comparison of sources of power to strike a proper level of load and hours of use which would be reasonably fair to all of them. Dr. Straus' hypothesis of one-third of the time at full load and two-thirds at half load is, perhaps, as reasonable, all things considered, as any guess is likely to be. Under these circumstances and with a total use of 3000 hours per year the several sources of power range themselves in the following order, beginning with the highest in cost: Steam engine, gasoline engine, benzol engine, locomobile, ordinary gas engine, Diesel engine, producer gas engine. Practically the same order is held down to 2000 hours per year of use. At 1000 yearly hours of use the ordinary gas engine and the benzol engine make the best showing. The range of costs per horsepower based on the use of 3000 hours per year varies from about 2 cents at the bottom of the list to about double that amount at the top. As compared with the cost of electric motive power the latter at 2½ cents a kilowatt-hour for all practical purposes surpasses anything in the way of a local prime mover that can be found, and when it comes to short time use, say 1000 hours per year, the motor leads, even at 4 cents per kilowatt-hour. For half this annual use the electric power is cheap at 6 cents and these figures are based on German prices for machinery and labor which are somewhat lower than our own.

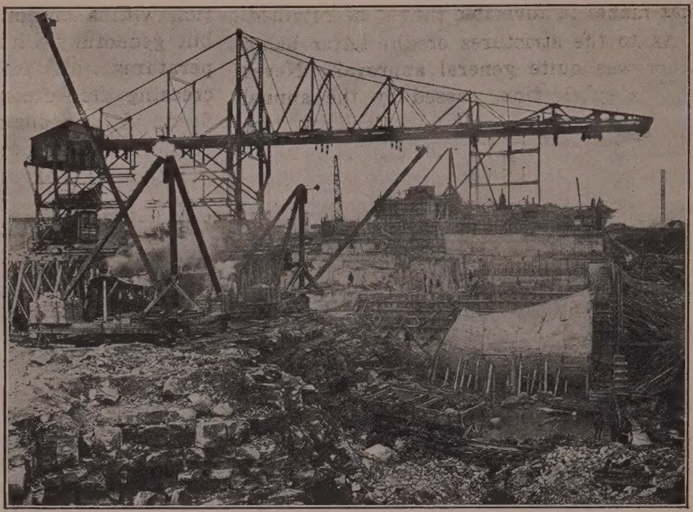
Of the prime movers considered, the producer gas plant cannot be considered in the category of a really portable affair and neither can the ordinary gas engine. Neither the Diesel engine nor the locomobile are extensively used in this country, and the benzol motor, on account of relatively higher cost of fuel in this country, has no particular advantage over the gasoline engine, which, under the most favorable circumstances is perhaps comparable with electric power at 3 or 3½ cents per kilowatt-hour, according to the hours of use per year and the price of gasoline, now certainly on the up-grade although the progress is somewhat irregular. In case of really intermittent use of a few hundred hours per year even 5-cent electricity is still cheaper than gasoline power.

Speaking broadly, therefore, in the small power unit of 20 hp the gasoline engine is easily the cheapest portable source of power where electric service is not available. The latter, when obtainable at a moderate price, shows greater economies than gasoline, and is more and more effective as the use becomes more intermittent and the average load lighter. For certain purposes the steam engine is almost preferable to gasoline on account of the ease with which the output can be forced to meet emergencies, and the little combination locomobiles, not yet known in this country, are considerably ahead of the ordinary steam engines.

All things considered, however, one who has to use portable sources of power can generally save money by using electric service wherever it is available. It will be well worth the while of anybody who is interested in cheap motive power to study the paper which serves as the text of this discourse, and to see for himself the effects of varying conditions of use and cost on the figures here abstracted.



POWER HOUSE WITH DRAFT-TUBE OPENINGS



TRAVELER AND FORMWORK FOR POWER HOUSE

Cedars Rapids Hydroelectric Development

Work on a 160,000-Horsepower Low-Head Plant Containing the Largest Sized Units in the World and Having a Headrace Two Miles Long

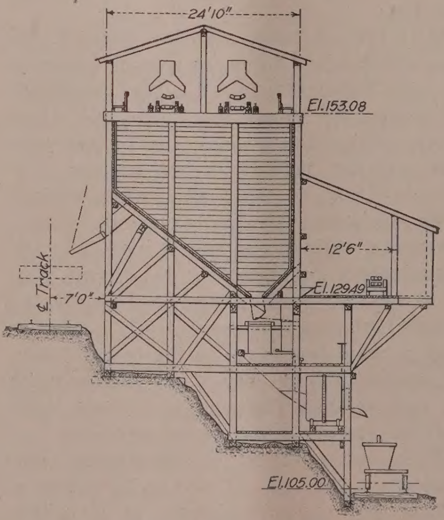
CONSTRUCTION of the Cedars Rapids Manufacturing & Power Company's 160,000-hp hydroelectric development near the village of Cedars and 30 mi. above Montreal has now proceeded far enough to permit the installation of the mechanical equipment. Already two of the turbines have been erected and two more are in course of erection. The stator of the first generator has been placed and parts of several more generators are stored in the yard. The design of this development was described in an article in the Engineering Record of Oct. 25, 1913, page 461, and no changes in design have been made since that time.

On the accompanying plan are shown the layout of the construction plant, the transportation facilities and the cofferdams necessary to unwater the site. From this plan it is apparent that the major portion of the excavation consisted in removing two large projections of the shore and straightening it to form the north bank of the headrace. The bed of the river between these points had also to be lowered from 2 to 10 ft. From the west projection of the shore to Isle aux Vaches no cofferdam was used for building the south dike of this canal. As this part is at the intake of the headrace it is subjected to only a small hydrostatic head and, consequently, no concrete corewall was considered necessary. It was, therefore, possible to construct this dike without unwatering the site and a special cableway carrying a dumping bridge was adopted. The major portion of the dike was specified to be made of rock and provided with an impervious earth blanket on the canal side. The slope is later to be riprapped. The accompanying photograph illustrates the method of depositing the materials in this portion of the dike. As the current was very swift at this point a certain part of the rockfill was washed away, but this had been expected when the method was adopted. The fill was taken directly from the excavations nearby. Only the rock portion of the dike was deposited from the cableway and afterwards the earth lining of the inside slope was dumped from the top of the rock embankment. Standard gage K & J 6-yd. cars were used for dumping the rock, while Western dump-cars of 12 cu. yd. capacity were used for the earth blanket.

COFFERDAMS

From the east end of the west projection of the shore all the way down to the power house a concrete corewall was specified for the dike. This in connection with the excavations in the river bottom necessitated the building of a crib cofferdam 8000 ft. long. This dike is now practically completed except the lower set of ice chutes located near the power house. The upper set of ice sluices, located at the western projection of the shore has been completed some time.

At the power house the cofferdams consist of rock dumps tightened with earth. The cofferdams have shown to be remarkably tight but there is considerable leakage through seams in the rock. Pumping is done at the power house site where four 10-in. centrifugal pumps are installed, two of which are capable of taking care of the leakage under normal conditions. These pumps elevate the water sufficiently to permit an elevated wooden flume to carry the discharge over the cofferdam. From three to five sumps have been in constant use at varying locations in the head and tail races. The number of pumps in each of these sumps has been varied as the work progressed.



ARRANGEMENT OF BINS FOR MIXER

As the construction of the north bank of the canal cuts off the drainage of a basin with a shore length of about 2 miles and extending back to the Soulanges Canal, it was necessary to intercept the runoff so as to prevent the flooding of that basin. A ditch was, therefore, dug parallel to the future canal and terminating below the power house cofferdam. The upper part of the ditch is 4 ft. wide at the bottom with side slopes 1:2. The lower part has a bottom width of 6 ft. and side slopes of 1:1½. Its bottom, which has a slope of 0.2 per cent, and 6 ft. of the side slopes are paved to prevent scouring. The deep portion of this ditch was excavated by means of a Lidgerwood dragline excavator loading the material into 12-cu. yd. Western dump-cars. The maximum rate of progress was 400 cu. yd. per day.

TRANSPORTATION OF MATERIALS

As may be seen on the general plans the site of the work adjoins the Soulanges Canal, a 14-ft. waterway. Nearly all material, machinery and supplies are received by way of this canal. The wharves are built on the main canal and along the headrace of the Provincial Light, Heat & Power Company. Provisions have been made to unload materials rapidly. The large quantities of materials received in barges are sand, cement, lumber and coal. As shipping on the canal opens in May and closes in November it was necessary to construct a cement shed of 35,000 bbl. capacity. This quantity was stored in 1913 and was used before May 16, 1914, so that the large capacity of this shed was appreciated. The cement is shipped in sacks, unloaded by an elevator and placed on a belt conveyor running into the shed. This conveyor is reversible so that it can also be used for loading sacks from the shed on to cars standing on a track between the shed and the unloading elevator.

The coal is unloaded by a clamshell bucket and deposited in a hopper feeding a

C. W. Hunt automatic railway for coal storage. The trestle has a coal hopper from which cars can be directly loaded. It was necessary to store about 15,000 tons of coal for use during the season of closed navigation. Sand is also unloaded by a clamshell bucket and placed on cars or in a pile. A portion of the sand, however, is produced by a pulverizer installed in the crusher plant.

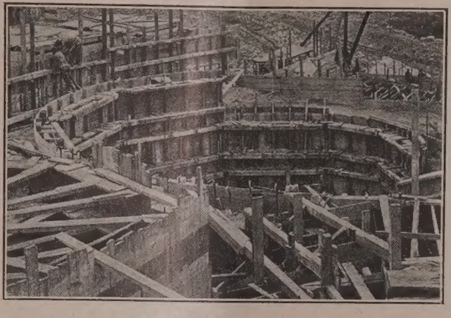
The Unit Construction Company, of St. Louis, which has the contract for building the power house superstructure and the transformer house, has located its yard at the north end of the site near the Soulanges Canal. Here the units are made and seasoned previous to their placing in the structures.

A trestle carrying crane girders and a 30-ton overhead travelling crane is used for unloading and storing the heavy parts for generators and turbines when they arrive and later on to place them on cars for transportation to the power house. In order to protect the crane from damage it is provided with a wooden housing. This crane will later be erected in the gate room of the power house.

As the generating station of the Provincial Light, Heat & Power Company is located only ¼ mile below the power house site electric energy is used as the main source of power on the work. Coal is used only for steam shovels, dragline excavator, locomotive cranes, locomotives, etc. Excavation was carried on all winter and it was necessary to keep the feed water mains from freezing. This was accomplished by means of an electric current run through the pipes, as described in the Engineering Record of April 4, page 399. Concrete work was discontinued during January and February.

CRUSHING AND MIXING PLANT

At a point about midway between the wharf and the power house is located the crushing and mixing plant. The initial crusher is a No. 7½ Traylor gyratory crusher from which a 24-in. bucket elevator lifts the product about 70 ft. to a revolving screen. This screen gives the following grading: Sand, including everything up to ⅜ in. size; stone, including sizes between ⅜ to 1½ in.; stone, including sizes from 1½ to 2½ in. Broken stone larger than 2½ in. is conveyed by a chute to two No. 5 Traylor crushers, which are also used as primary crushers, and by a 20-in. belt conveyor fed back to the bucket elevator. The graded sizes from the screen are distributed by belt conveyors and travelling trippers in the bins above the mixers. From these bins chutes discharge the aggregates into measuring hoppers.



ARRANGEMENT OF SCROLL-CASE FORMS

Cement is taken from the store house at the wharf and placed in a shed at the mixing plant. An 18-in. belt conveyor carries the sacks from this shed to the cement gallery alongside the mixer bins. A specified number of sacks per batch is dumped from this gallery into the measuring chutes.

A pulverizing plant has been installed in connection with this plant consisting of a No. 4 Williams pulverizer running at 1000 r.p.m. Its maximum output has been about 200 cu. yd. per day, which seems to be the limit capacity of the machine under these conditions. The parts of the machine wear out quickly, but they are easily replaced. There are sixty-six hammers, which on an average are worn out after three or four days service. Sometimes a set of hammers is worn out after one day, occasionally they last ten days. The rock used is a hard, blue lime-stone containing a high percentage of silicon which puts high demands on the crushing and pulverizing machinery. Manganese-steel hammers have, therefore, been tried and although they last longer, they do not make as much sand as hammers of low-carbon tool-steel. The lining of the pulverizer stands the wear about four to five days, while the grid is worn out after one week. These figures refer to the machine when its output is 200 cu. yd. per day.

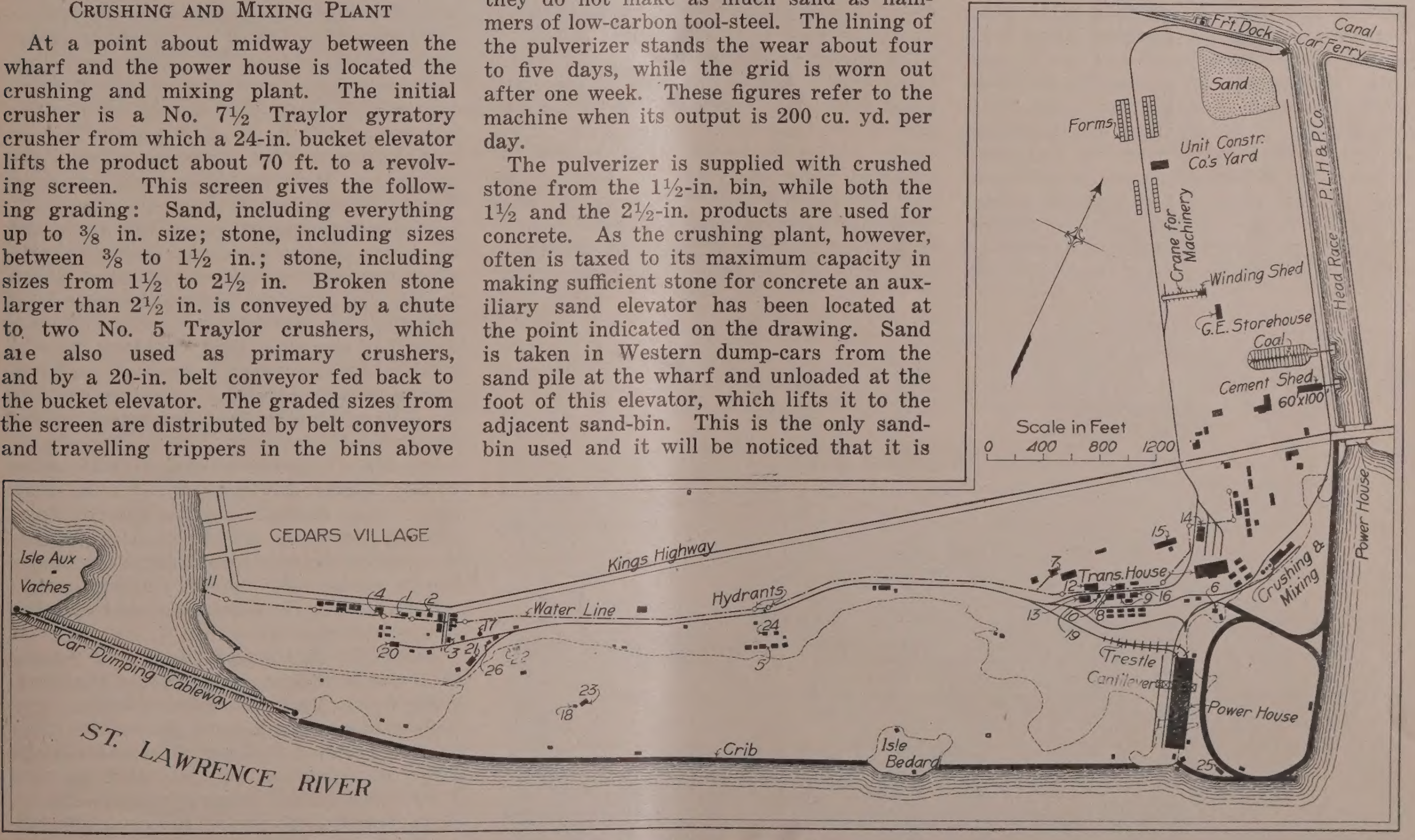
The pulverizer is supplied with crushed stone from the 1½-in. bin, while both the 1½ and the 2½-in. products are used for concrete. As the crushing plant, however, often is taxed to its maximum capacity in making sufficient stone for concrete an auxiliary sand elevator has been located at the point indicated on the drawing. Sand is taken in Western dump-cars from the sand pile at the wharf and unloaded at the foot of this elevator, which lifts it to the adjacent sand-bin. This is the only sand-bin used and it will be noticed that it is built to serve two mixers at the same time.

Originally two No. 5 Smith mixers and two Chain Belt mixers of 1½-cu. yd. capacity were installed, but the contractors found the two Chain Belt machines to be sufficient. These two mixers have turned out 550 batches or about 600 cu. yd. per 10-hour day. The entire plant is driven by electric motors shown on the accompanying drawing. Near the mixing plant is a little shed in which torn cement sacks are mended. As no torn bags are redeemed the contractors bought a sewing machine and now 135 to 160 sacks are patched or darned per day.

For placing concrete in the corewall of the dike a portable ½-cu. yd. Smith mixer was used. This mixer was mounted on a flat car and the guides for the charging bucket were extended to the ground. From the mixer the batches were elevated in a Ransome bucket and tower also mounted on the car and deposited in the forms.

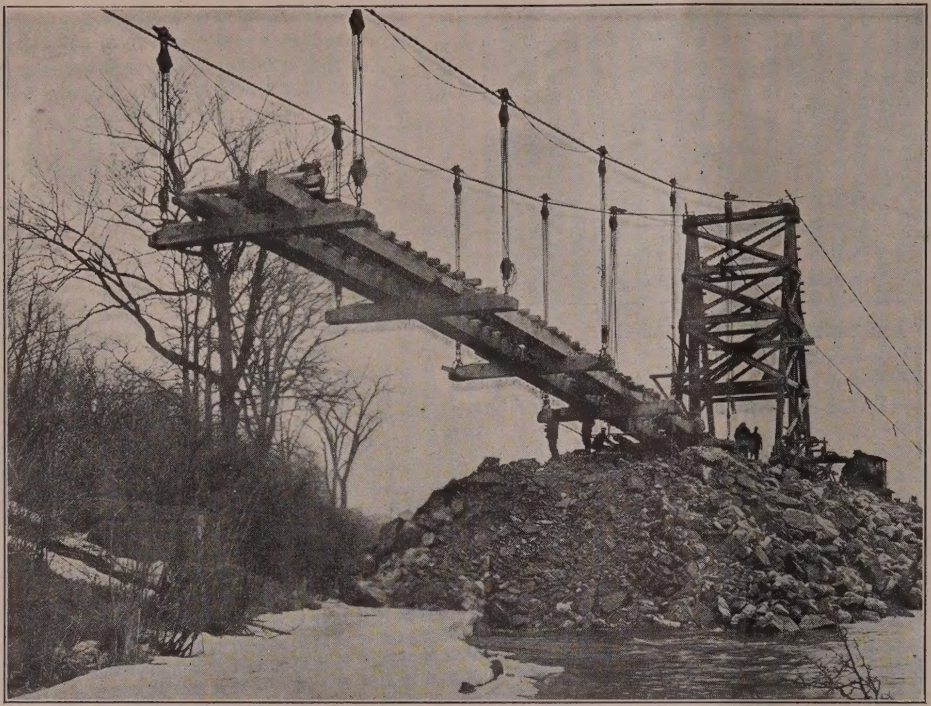
POWER HOUSE SUBSTRUCTURE

The design of the power house substructure was shown in the Engineering Record of Oct. 25, 1913, page 461. To facilitate the erection of forms and the placing of concrete a 150-ft. cantilever traveler with three trolley beams is used. This traveler was not used for the excavation, which was done by means of a Bucyrus steam shovel and four 10-ton derricks. The traveler is the one formerly used at Keokuk for building the dam across the Mississippi River and it was described in the Engineering Record of Aug. 5, 1911, page 148. It is, however, used in a different way and it has been provided with a new substructure raising it 25 ft. Instead of being placed in the direction of the concrete supply track, thus making the path of the bucket linear,



PLAN SHOWING THE SITE AND CONSTRUCTION PLANT FOR THE CEDARS RAPIDS DEVELOPMENT

1, Compressor house; 2, Blacksmith shop; 3, Store house; 4, Temporary hospital and doctor's office; 5, Office building; 6, Field office; 7, 5000-gal. water tank on 50-ft. steel tower; 8, Carpenter shop completed; 9, Blacksmith shop; 10, Machine shop; 11, Pump house; 12, Plant store house; 13, Engine house; 14, General store house; 15, Carpenter shop; 16, Compressor house; 17, Water tank and heater room; 18, Pump house; 19, Water tank and heater room; 20, Engine house; 21, Cement shed; 22, Blacksmith shop; 23, Boiler house; 24, Boiler-house heating plant; 25, Pump house; 26, Stone crushing plant.



CABLEWAY DUMPING BRIDGE FOR BUILDING UPPER PART OF CANAL DIKE

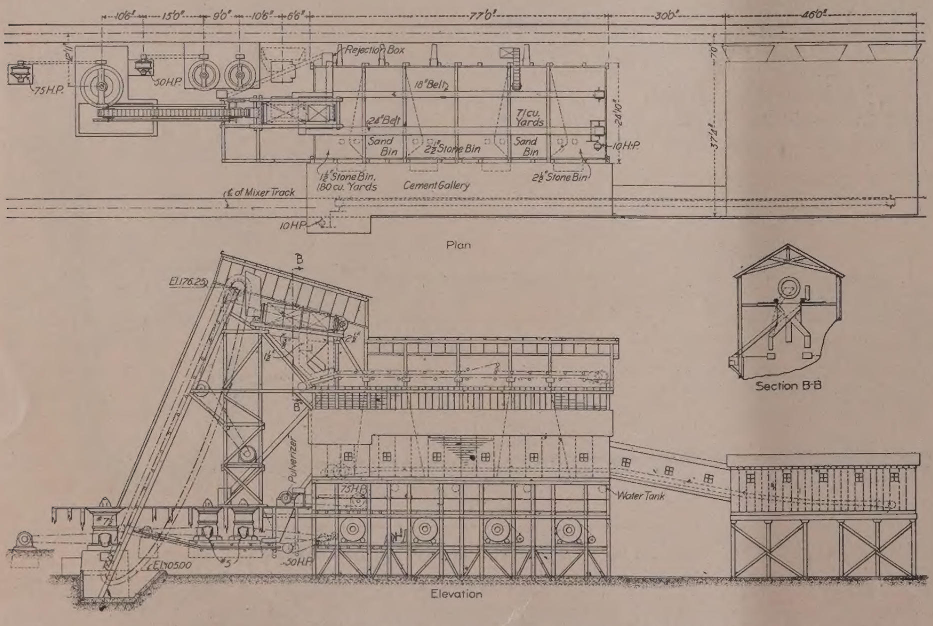
it is now placed at right angles to it. At Keokuk the six Lidgerwood hoisting and traversing engines were driven by compressed air. Sufficient air not being available on this job, a locomotive boiler had to be installed in the lower part of the new addition.

After the excavation was completed concrete was deposited below the draft tubes so as to form a seat for the form work. The draft tube forms were designed so that they could be collapsed, withdrawn and used over again. The various parts were, therefore, aligned on a central rib. Each form was used three times and one of them four. Expansion joints in the concrete were provided in the center of the piers between units. The concrete was deposited around the draft-tube forms and the joints between the different courses so arranged that they are as nearly as possible normal to the curvature of the opening formed. The neck of the draft-tube is carried up above the scroll case floor of the unit, forming a seat on which the speed ring of the unit is erected.

After the floor of the scroll case has been finished the forms for its walls are erected. These forms are also built in units in such a way that no interior bracing is necessary for the spiral itself. This is not true, however, for the intake to the spiral where the forms are heavily braced. All forms, where possible, are designed for a 6-ft. lift of wet concrete. The mixture is generally 1:3:6 except in places where a richer mixture is necessary, as, for instance, in reinforced-concrete slabs and beams.

To facilitate the erection of the unit a large hole is left in the roof of the scroll case. The roof, therefore, lacking the support it will have when completed, is temporarily supported on six 24-in. square columns of plain concrete. As soon as the unit is erected and the concrete work completed these columns are knocked out.

To facilitate the erection of machinery the frame of the power house superstructure is made of structural steel. This permitted quick erection of crane girders and cranes, of which there are two 150-ton in the generator room and one 30-ton in the gate room, made by the Whiting Foundry Equipment Company. The machinery is brought into the power house over two trestles, one leading up to the higher level of the gate room and one in on the generator floor.



PLAN AND ELEVATION OF CRUSHING, PULVERIZING AND MIXING PLANT

The progress of the work has been very rapid as indicated by the following progress reports:

WORK COMPLETED, PER CENT

	Aug. 31, 1913	Dec. 1, 1913	May 1, 1914
Rock excavation	21	23	58
Earth excavation other than stripping and trench work	43	60	73
Earth excavation in trenches and ditches and stripping of seats of banks	36	84	*
Transporting and placing excavated rock	20	28	62
Stone protection	2	3	5
Transporting and placing excavated earth	43	63	77
Concrete in power house substructure	16	26	69

*Completed.

The total quantities are about 100,000 cu. yd. concrete, 650,000 cu. yd. rock excavation, and 1,800,000 cu. yd. earth excavation.

PERSONNEL

The development has been designed by the engineering department of the Cedars Rapids Manufacturing & Power Company, and is being constructed under the supervision of the company. Julian C. Smith is hydraulic engineer and in charge of the design of the hydraulic end of the development and in direct charge of all of the construction work, except the electrical work. R. M. Wilson is electrical engineer and in charge of the electrical design, and the construction work relating to the electrical installations. Alexander Wilson is assistant in charge of the design work for the electrical part of the development.

Representing these men are the resident engineer of the plant, George P. Hawley, and his assistant, H. S. Slocum. The designing was carried out in the office of the company in Montreal under the direction of Messrs. Smith and Wilson.

Under the supervision of the above and in direct charge of C. S. Saunders, construction superintendent of the Shawinigan Company, the cofferdams were built and the site unwatered during the year 1912, and the early part of 1913. During this time also a large amount of construction plant was purchased and installed and the general layout of the construction plant completed.

On May 1, 1913, a contract was let to Fraser, Brace & Company, of New York City. This contract covered the construction of the power house substructure, excavation in the canal and practically all the work except the power house superstructure, transformer house and installation of the apparatus.

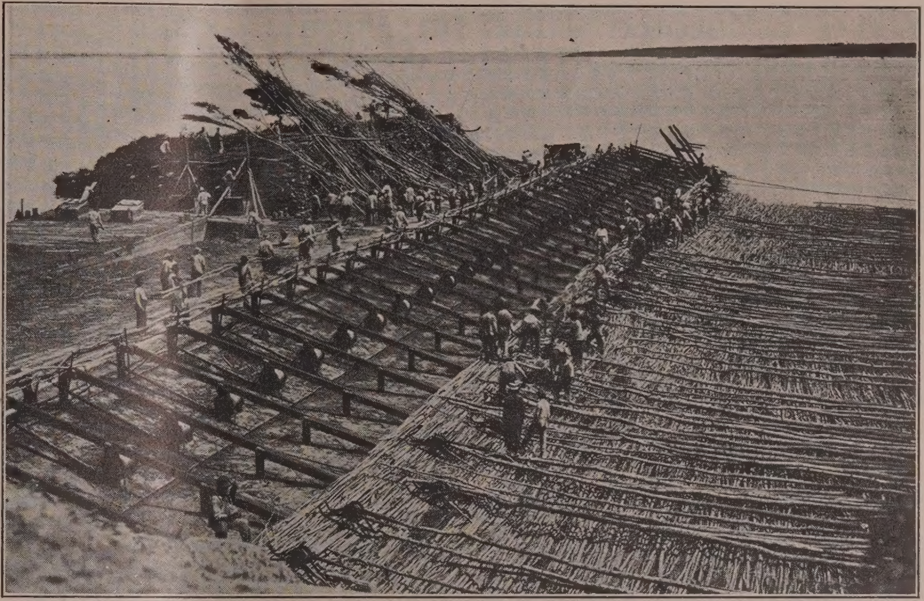
James H. Brace and Charles E. Fraser both live at the site, supervising the work, while G. C. Clarke of the contracting company also devotes a large part of his time to the work at Cedars. The contractors' large organization on the work includes J. J. McCarthy, superintendent; N. J. Kayser and C. H. Covey, assistant superintendents; John Menzies, master mechanic; G. A. Bullock, chief electrician, and Fred. Merchant, formerly assistant superintendent at the Keokuk development, is in charge of the carpenter work.

Mississippi River Protection Mat Construction

Building and Sinking 1,250,000 Sq. Ft. of Willow Mat to Prevent Formation of Cutoff by Erosion of River Banks

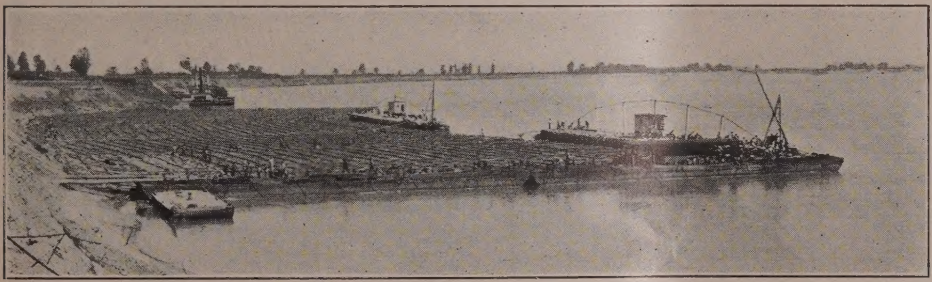
THE prevention of erosion on large areas of the bottom and banks of the Mississippi River by the construction of willow mats is illustrated by work of this character executed in 1913 by the United States Government to prevent the threatened formation of a cutoff at Slough Landing Neck. This work was of typical nature, done by standard methods and plant.

The protection consists of a stone and willow mat covering the sloping sides and bottom of the Mississippi River for a width of 250 ft. and a length of 5000 ft. up and downstream. It was built at a total cost of about $175,000 and involves about 28,000 cu. yd. of stone and 19,000 cords of willow brush. It was built in full width sections about 1000 ft. long, secured at the upper end and floated until the weaving was completed. It was ballasted and sunk at an average rate of about 100 ft. per day by a force of 200 men operating a $250,000 plant.



MAT BARGE AFTER A LAUNCH; WAYS AND DRUMS READY FOR FASCINES

The mat consists essentially of a continuous mass, about 12 in. thick, of long slender willow poles transverse to the axis of the river, bound together with small wire ropes and covered with broken stone deposited in 8 x 8-ft. shallow pockets formed by longitudinal and transverse poles. The mat is constructed on long barges securely moored transverse to the river with one end close to shore. A mat head consisting of a bundle of poles 3 ft. in diameter is first built and is followed by the construction of the regular mat made up of parallel fascines about 12 in. in diameter attached to it. The operations are performed on inclined launching ways extending the full length of the barge. The fascines are bound with ¼-in. galvanized iron wire rope stressed by a drum-and-ratchet lever located near the top of the inclined ways. The fascines are rolled down the ways to the edge of the mat and sewn together with ¼-in. galvanized iron rope, and are attached to longitudinal top and bottom lines of ½ in. and 5/16 in. galvanized iron wire rope 8 ft. apart transversely, connected at intervals by clips and vertical wires called anchor stops. These ropes run continuously from end to end of each section of the mat and are supplemented at intervals of 8 to 15 ft. by parallel lines of 9-in. silicon bronze wire, designed to be sufficient to hold the mat together in case the galvanized iron wires are corroded to the point of failure.

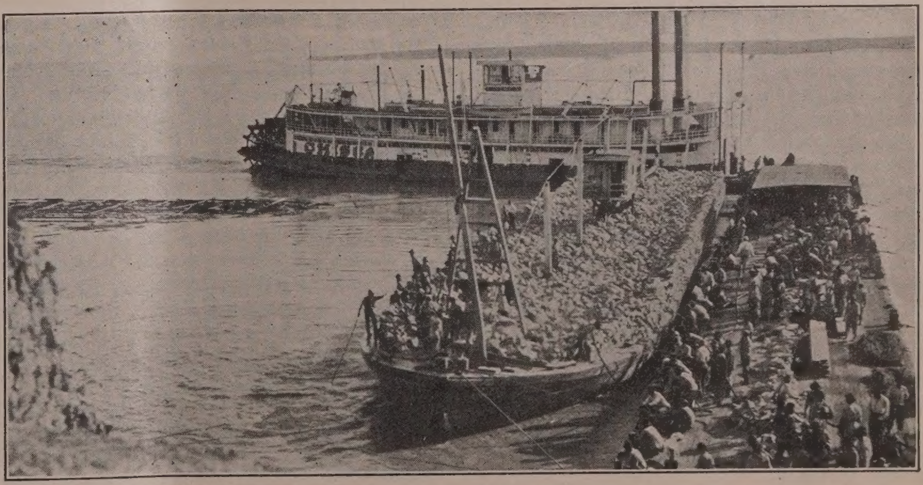


POLING THE MAT TO FORM POCKETS FOR RECEIVING BALLAST

As the ways are filled with fascines and the latter are tied together, the mooring cables are slacked off, allowing the weaving barge to move downstream, receding from the anchored upper end of the mat, and leaving the ways free for the continual addition of fascines. The mat is anchored by 1⅛-in. wire cables at the upstream end, and is built at an average rate of 80 to 100 ft. per day. The upper surface is divided into 8 x 8-ft. pockets for ballast and after the weaving is completed the mat is loaded with broken stone distributed over the surface by wheelbarrows. The mat barge is then swung out and disconnected from the foot of the mat, and the mat is sunk by throwing additional ballast on from the stone barge. As the mat sinks the barge is allowed to drift downstream over it, depositing more ballast and sinking the remainder of the mat to its permanent position on the bottom of the river at the rate of about 1000 lin. ft. in three days.

In order to provide a suitable uniform slope for the longitudinal shore edge of the mat the shore is graded where necessary to a 3:1 slope by a hydraulic nozzle operated with a pressure of 225 lb. per square inch derived from a pump on the attendant barge. Above low water this slope is paved with 9 in. of riprap or with 4 in. of 1:12 concrete made of river sand and gravel and laid in strips 6 ft. wide. The plant required for this construction included one steamboat, one hydraulic grader, three quarters boats, two mooring barges, two weaving or mat barges, two fascine barges, six willow barges, eight stone barges, and one line barge or flat, representing a total value of about $250,000. The work was executed under the direction of Major E. M. Markham, Corps of Engineers, U. S. Army; W. M. Gardner was principal assistant in charge.

CONCRETE STREETS and highways in California laid during 1913 total 1,776,000 sq. yd., according to statistics recently announced by the California State Highway Commission. When the $18,000,000 State highway system is completed it is believed that California will have a greater area of concrete roadways than New York State, which now holds first place.



SUBMERGING HEAD OF MAT, WITH RIPRAP UNLOADED BY HAND

Creosoted Piles on Pacific Coast

Résumé of Interviews with Men of Large Experience in Building and Maintaining Timber Structures in Various Western Ports

By N. A. BOWERS
Pacific Coast Editor, Engineering Record

"TWENTY-FOUR years in service and no renewals." This is the record of a creosoted-pile freight wharf at Oakland, Cal., which has an area of 630,000 sq. ft. and is now handling an average of 150 cars of freight per day. Perhaps this is the best example of long life being pointed out by Western advocates of creosoted-pile construction, but it is not unique, nor is this length of life considered the maximum. However, engineers do not all agree that this is the most economical material for piles, as is proved by the large amount of concrete wharf and dock work under way between Vancouver and Los Angeles. In fact, it is difficult to gather opinions from those qualified to speak without being drawn into a discussion on the comparative merits of timber and concrete. It is worth while, however, to disregard the much-mooted questions long enough to analyze the experience with creosoted piles in Western ports, which has developed so much confidence in them there in recent years.

Remarkably short life is attributed to untreated Douglas-fir piles in San Francisco and southern California ports, about 2½ years being considered by some engineers the longest that should be expected. When untreated piles are exposed to abrasion, as in the fender line, the life will average less than a year, it is said, due partly to wear, of course, but more directly to the fact that when the pile suffers external injury the action of borers is greatly accelerated. Other engineers who have had more satisfactory experience with the untreated piles place its probable average life in service, other than in fender lines, at from 4 to 6 years. It is significant that 100 per cent of all the untreated piles driven since 1906 by one railroad company in four Pacific Coast ports had to be replaced within 7 years. It is, therefore, generally agreed that some form of treatment is necessary if wooden piles are to be used in the more permanent structures; and, after considerable experiment, treatment with creosote has come to be the only method now used extensively on the Pacific Coast.

COST OF CREOSOTING PILES

The question as to whether it is economical to continue treating piles, now that creosote costs 10.04 cents, as against 7¼ cents per gallon in 1912, was recently sent from the manager's office to the engineering department of a large company. The engineer showed in reply that a 1-cent increase in the price per gallon of creosote would increase the cost of the treated piles 1 cent per linear foot, and that, taking the life of untreated piles at 8 years and creosoted piles at 30 years, it would be economical to treat piles until the cost of treatment reached 32 cents per linear foot, which corresponds to a creosote cost of 22 cents per gallon. These figures are based on a cost for untreated Douglas-fir piles of 10 cents per foot, as against 30 cents for creosoted piles. Assuming $6 for driving in each case, a 40-ft. untreated pile in place would be worth $10 and a treated pile of equal length in place would cost $18.

Table 1 shows average treatment costs on the piles creosoted by the Southern Pacific Company for the last eight years. It should be said in connection with this table that the fluctuation in cost of treatment is partially due to the number of piles handled as well as to the variations in the price of creosote. No attempt was made to include the cost of the piles, because of the rapid fluctuations in their cost. In April, 1914, Douglas-fir piles brought 11 cents per linear foot in San Francisco, while in the following month the price dropped to 6.8 cents. As a rule, from 6 to 10 cents per linear foot is the normal range at San Francisco for lengths under 50 ft., and this increases up to 28 cents on lengths of over 110 ft.

A committee of three engineers appointed by the San Francisco Chamber of Commerce to make an inspection of harbor structures in that port mentioned in its report that good creosoted piles should have a useful life in San Francisco Bay of at least 20 years. Records in the offices of the Southern Pacific Company show that in this port about 8 per cent of the creosoted piles in service for 20 years have had to be renewed for causes other than mechanical breakage, and the remaining 92 per cent are said to be in such good condition that a total life of at least 30 years is expected. Records covering 16,000 creosoted piles driven on land on the Los Angeles division of this railroad show that the average length of service is now 17 years and the maximum 21 years, and that out of this total number of piles, 269, or 1.6 per cent, have decayed sufficiently to warrant renewal.

TABLE 1—UNIT CREOSOTING COSTS FOR EIGHT YEARS

Year	Creosote per gal., cents	Creosote per lin. ft., cents	Total treatm't cost per lin. ft., cents
1907	8	7.6	14.6
1908	7.5	6.8	16.6
1909	8.04	7.6	18.8
1910	7.69	7.0	15.1
1911	8.28	8.0	14.7
1912	7.25	7.1	17.1
1913	8.25	8.1	17.7
1914	10.04	9.8	19.0

The life of a treated pile, as pointed out by a number of engineers independently, will be determined largely by the condition of its outer surface after it is driven. Mechanical injuries, such as result from hooks, spikes or hammer blows, are to be carefully avoided, but even more serious than these is the checking that may result from treatment or that may develop in driving. In addition to the weakening and the exposure to the teredo due to checking, a new danger was discovered recently in Wilmington harbor. In this case piles which had checked considerably were found to be fairly sound on the surface, but within had been cut into "lacework" by borers much smaller than the teredo. Sections of these piles were submitted to experts to determine whether some hitherto unknown marine borer had attacked them, but careful examination showed the borers to be the common limnoria, which had abandoned its usual method of cutting around the exterior and had gone inside through cracks in the otherwise well-protected surface.

On the other hand, an instance of exceptionally good surface protection is found in the long wharf at Santa Monica. Piles in this structure were treated and driven in 1892 and at present the outer end of the wharf is no longer in use and is being removed by breaking the piles off at the ground line, about 55 ft. below the surface. After 22 years of service most of the piles removed are being sharpened and redriven in the repair of bents beneath an electric railway track, and the foreman of the redriving crew reports them less liable to break under the hammer than freshly treated piles.

In looking up the history of these particular piles it developed that prior to the building of this wharf there was some question in the opinion of army engineers whether it should be built at Santa Monica or San Pedro. Meanwhile the piles lay on shore, pending this decision, long enough to become well seasoned before they were treated. At that time the Bethell creosoting process was in vogue, but this was found to be inapplicable to Douglas-fir piles or timber, and was abandoned for this piece of work except for a few bents at the inshore end. The boiling process described below was adopted for the rest of this wharf.

METHODS OF TREATMENT

Now that the experimental stage of timber treating has passed, the only differences in practically all the treating plants on the Pacific Coast are in the method of preparing the timber and in the quality of creosote injected. The specifications for creosote approved by the American Railway Engineering Association are accepted as standard, except by the Southern Pacific Company, which has its own specifications. Some form of boiling process is now used almost exclusively in the treatment. The boiling process was developed in 1892 by John D. Isaacs for the Southern Pacific Company, which treats a large percentage of the piles used in central and southern California. The process became generally known, and at present five of the six timber-treating plants in Oregon and Washington employ it.

In this process, which was designed primarily for treating Douglas fir, the artificial seasoning by boiling in oil is said to weaken the timber much less than the steaming process. After the piles are run into the treating cylinder on trucks, creosote at a temperature of 160 deg. Fahr. is introduced until the piles are covered and there is a space of 10 in. between the surface of the oil and the top of the retort. Steam is then admitted to coils in the cylinder and the temperature of the oil is raised to 220 deg. The amount of steam is then regulated so as to maintain the temperature between 220 and 225 deg. until such time as the amount of condensation collected from the treating cylinder has decreased to ⅛ lb. of water per cubic foot of timber per hour. After this the treating cylinder is filled with creosote at a temperature of 180 deg. and a pressure of 5 lb. per square inch is developed. Connection is then made with a measuring tank from which oil is pumped into the cylinder until an amount equal to 10 lb. for each cubic foot of timber has been injected.

PIER CONSTRUCTION AND MAINTENANCE COSTS

Creosoted-pile piers of the construction standard approved for the port of San

Francisco by the Board of State Harbor Commissioners are usually estimated at $1 per square foot of completed pier, exclusive of bulkhead wall and sheds. This allows a margin for fluctuating prices of materials, as is shown by the fact that contracts for two piers have recently been let on a basis of 84 and 81.4 cents per square foot. On the basis of a cost of $1 per square foot the maintenance estimate in Table 2 was submitted to the San Francisco Chamber of Commerce by a committee of three engineers appointed to investigate dock costs and governing conditions at the port of San Francisco.

TABLE 2—TOTAL ANNUAL CHARGE AGAINST CREOSOTED PILE PIERS

Interest charges at 4 per cent	$0.0400
Annual maintenance	.0500
Replacement fund, 20 years	.0336
Redemption fund, 75 years	.0022
Insurance	.0060
Total annual charges per square foot	$0.1318

In recent years an increasing number of pleasure piers have been constructed on the Pacific Coast, particularly in the south. These piers, being of much lighter construction and requiring no provision for railroad tracks, are built at a much lower cost than those referred to above. In a typical pleasure pier built this year at Venice, Cal., the piles are spaced 10 ft. between centers one way and 15 ft. the other; 12 x 12-in. caps and 3 x 12-in. stringers are used and 4 x 6-in. bracing both ways is specified. Floor joists are bridged every 7½ ft. and a 2-in. floor is laid. Caps are drifted with 1 x 18-in. bolts and ½ x 18-in. drift bolts are used on stringers. This pier was built under contract at a cost that works out at 35 cents per square foot, although it should be said that about 50 cents has been the more common figure in previous years.

It is apparent, therefore, that aside from any comparisons with concrete construction, which it was not proposed to take up here, creosoted piles can be expected to give satisfactory service under Pacific Coast conditions. Moreover, sufficient time has, probably, not elapsed to allow final judgment to be passed in the controversy between timber and concrete under these conditions, for, as above noted, piles which have been removed were giving good service after 22 years of use, while others are still in good condition after a period of 24 years.

Water Consumption in Metropolitan District, Boston

Statistics Showing Per-Capita Figures for Eighteen Municipalities During 1913

WATER consumption in the eighteen municipalities comprising the metropolitan district in and near Boston averaged 94 gal. per capita daily during 1913. The total daily average consumption, 103,847,700 gal. as measured by Venturi meters, was 12,383,000 gal. less than during the previous year, somewhat less than during any year since 1901, and less per capita than in any year since 1895. The large reduction in the quantity used as compared with that of the previous year was due, according to the annual report of Dexter Brackett, chief engineer, to two causes—the more general use of meters and the unusually mild weather during the winter of 1912-1913. The consumption during the summer months was from 5,000,000 to 12,000,000 gal. less than for the corresponding time in the previous year, while the consumption during January and February was about 25,000,000 gal. per day less in 1913 than in 1912. The average temperature of the air for these winter months was 32 deg. in 1913 as compared with 22 deg. in 1912.

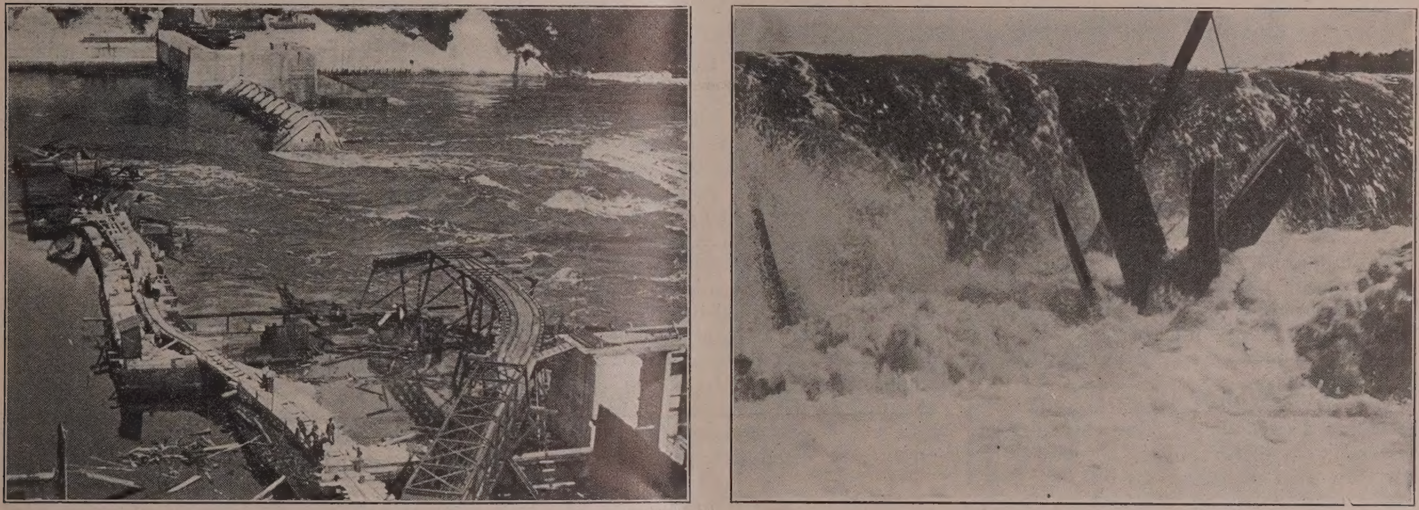
The average daily rates varied from 39 gal. in the town of Milton to 123 gal. in the town of Nahant, but the per capita use in Nahant is very large during the summer, for the reason that the town has a very large summer population, for which no allowance is made in the census population, upon which the per capita consumption is based. The average daily per capita consumption in the cities of Boston and Medford and the towns of Winthrop and Stoneham for the year 1913 was less than the average rate of use in the same municipalities between the hours of 1 and 4 a. m. during the year 1908, indicating that the total daily use is now less than the use and waste during the night hours before the more general introduction of meters.

From 1890 to 1902 the quantity used in the district now supplied from the metropolitan works increased very rapidly, and it seems probable that if no means had been adopted to restrict the waste of water, the consumption of the metropolitan district would, at the present time, be 60,000,000 gal. per day greater than it now is, and that the district would be expending several million dollars in procuring more water.

Mississippi River Cofferdam Failure

Extent of Damage Done at Lock and Dam 1, near Minneapolis, Shown by Photographs

A SHORT account of the failure June 29 of the lower cofferdam at the Mississippi River lock and dam 1, near Soldiers' Home, half way between Minneapolis and St. Paul, was given in the Current News of the Engineering Record of July 11, page 16. The accompanying photographs give an idea of the damage done. The account of last week was accompanied by a diagram showing the portions of the cofferdam that were washed out.



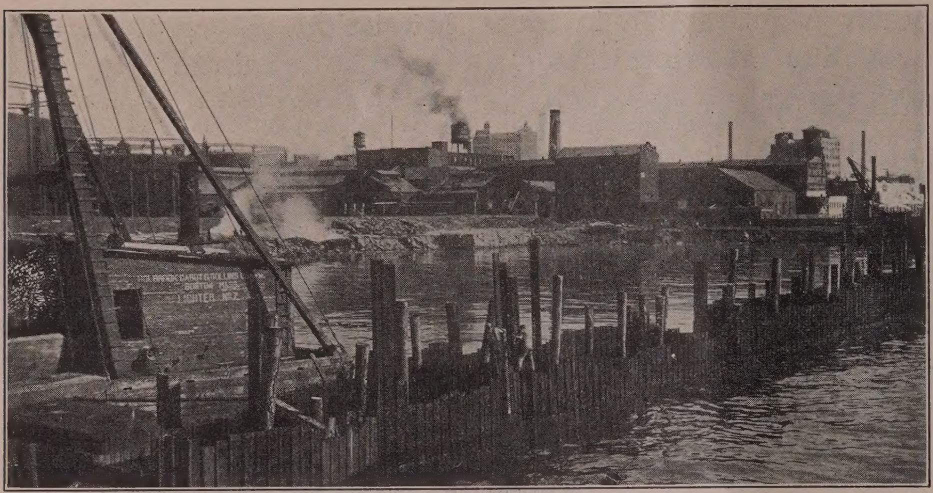
TWO VIEWS AT THE SCENE OF THE MISSISSIPPI RIVER COFFERDAM AT MINNEAPOLIS WHICH WAS WASHED OUT RECENTLY

Permanent Drainage Ditches

EXPERIENCE in the Little River drainage district of Missouri, where 624 mi. of open ditches are now under construction for the reclamation of about 488,000 acres of alluvial land, indicates that if the ditches are excavated with deep banks in sandy soil, soon after the bankful stage has receded to a lower depth and has relieved the head which sustained the bank, the latter will slough to about 1:1½ slope, and the corresponding increase in the depth will not exceed 2 ft. In a U-shape ditch difficulty is caused by caving, that is, by material falling in or tumbling over, and not by the bank sloughing or sliding as a unit on the slope.

In a ditch of the latter shape a crack will be formed on the berm, 4 or 5 ft. back from the edge extending to the normal water line ditch, and on that line the bank will creep back into the ditch. The vertical plane of this caving will adjust itself to its normal slope. The material will then fill the ditch to about 5 ft. and leave a wide shallow trench, the change being evident six months after the ditch has been built.

TANNERIES AND GLUE WORKS in the metropolitan sewerage district of Boston all have settling tanks of substantial size. This method of treatment, according to the 1913 annual report of Frederick D. Smith, engineer of sewerage works, has very greatly reduced the amount of sludge material entering the metropolitan sewers and has materially lessened the cost of maintenance of the sewers in this district.



POCKETS IN DEEP WATER ON RIVER SIDE OF COFFERDAM

Deep Cofferdam for 1050-Foot Pier in New York

Unbraced Steel Sheet-Pile and Riprap Construction of Pocket Type to Resist Maximum Pressure Head of 68 Feet

ONE of the deepest unbraced cofferdams, if not the deepest, of large dimensions that have ever been constructed is now being built of steel sheet piling in the Hudson River at West Forty-sixth Street, New York City, to provide for rock excavation and substructure work for a 1050-ft. steamship pier and deep slips adjacent. The construction was adopted to secure, without bracing, independent stability against an unbalanced maximum-pressure head of 68 ft.

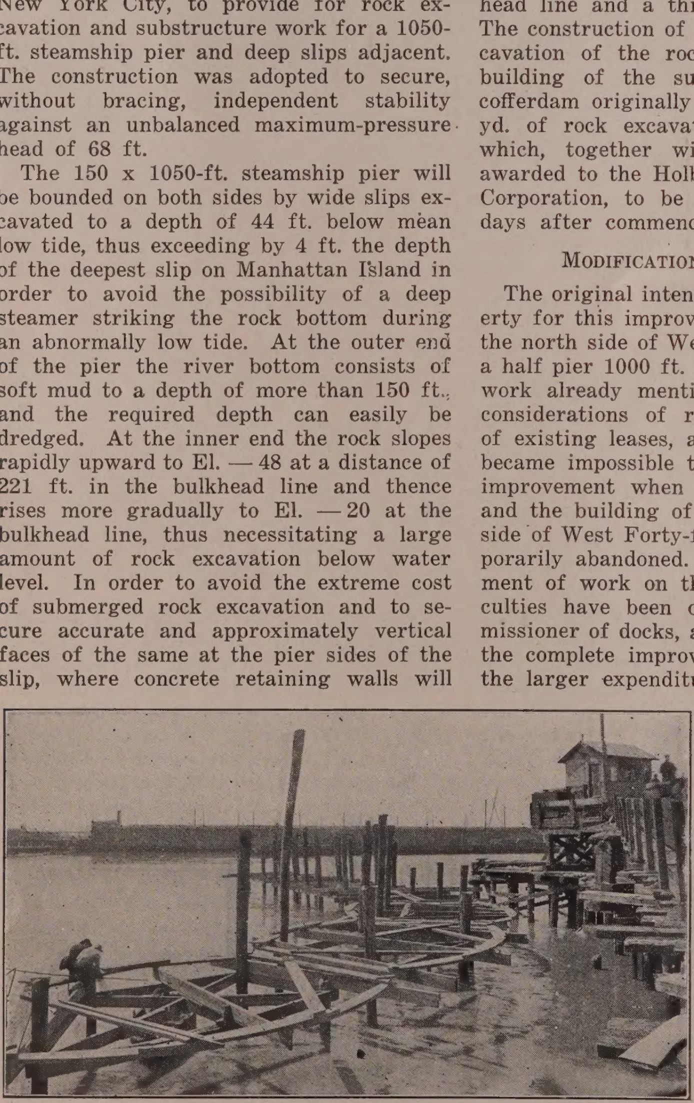
The 150 x 1050-ft. steamship pier will be bounded on both sides by wide slips excavated to a depth of 44 ft. below mean low tide, thus exceeding by 4 ft. the depth of the deepest slip on Manhattan Island in order to avoid the possibility of a deep steamer striking the rock bottom during an abnormally low tide. At the outer end of the pier the river bottom consists of soft mud to a depth of more than 150 ft., and the required depth can easily be dredged. At the inner end the rock slopes rapidly upward to El. — 48 at a distance of 221 ft. in the bulkhead line and thence rises more gradually to El. — 20 at the bulkhead line, thus necessitating a large amount of rock excavation below water level. In order to avoid the extreme cost of submerged rock excavation and to secure accurate and approximately vertical faces of the same at the pier sides of the slip, where concrete retaining walls will be built to sustain the earthfill carrying the pier floor, it was decided to inclose a 300 x 750-ft. area at the shore end of the pier and slip by a rectangular cofferdam with two sides perpendicular to the bulkhead line and a third side parallel to it. The construction of this cofferdam, the excavation of the rock and earth and the building of the substructure inside the cofferdam originally involved about 50,000 yd. of rock excavation, the contract for which, together with other items, was awarded to the Holbrook, Cabot & Rollins Corporation, to be completed within 550 days after commencement last March.

MODIFICATION OF CONTRACT

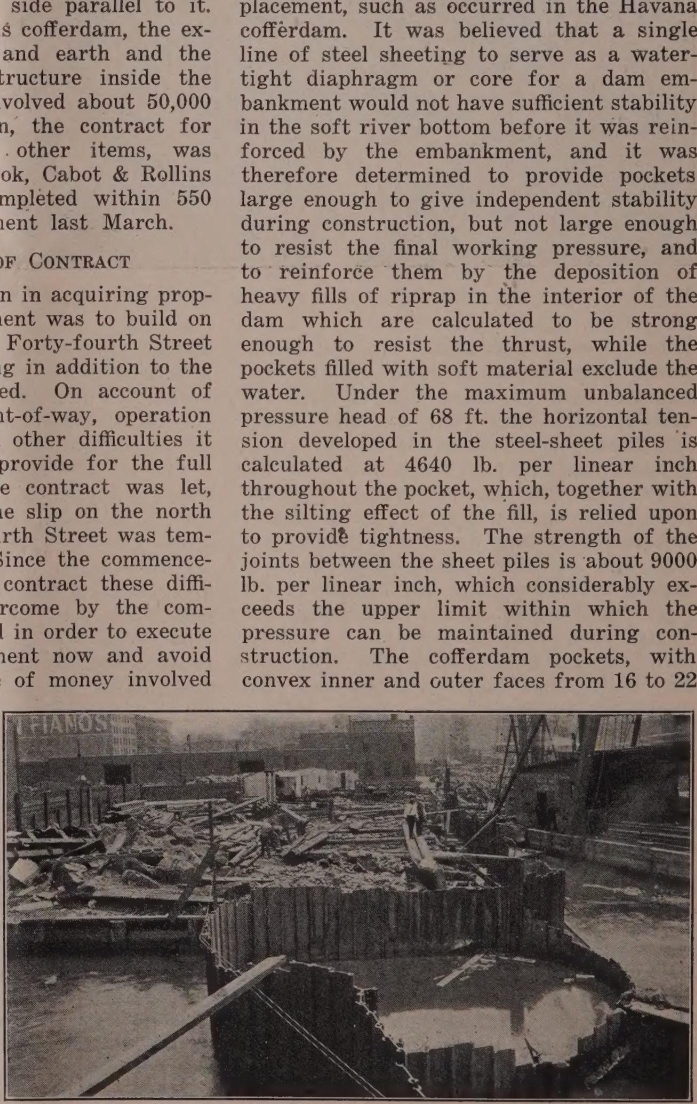
The original intention in acquiring property for this improvement was to build on the north side of West Forty-fourth Street a half pier 1000 ft. long in addition to the work already mentioned. On account of considerations of right-of-way, operation of existing leases, and other difficulties it became impossible to provide for the full improvement when the contract was let, and the building of the slip on the north side of West Forty-fourth Street was temporarily abandoned. Since the commencement of work on the contract these difficulties have been overcome by the commissioner of docks, and in order to execute the complete improvement now and avoid the larger expenditure of money involved by doing it at a future time, when extra work would be required, a modification of the contract has been accepted whereby the cofferdam will be extended to the north line of West Forty-fourth Street, entailing an additional expenditure at original unit prices of about $140,000 in excess of the original contract. The amended contract will, therefore, comprise the placing of about 3500 tons of steel sheet piles in the cofferdam, 99,000 yd. of riprap in the embankment, the removal of 76,000 yd. of solid rock and the construction of 14,000 yd. of concrete masonry. The basin inclosed in the cofferdam will cover an area 800 ft. long and 300 ft. wide and its unwatering will require the removal of about 55,000,000 gal. exclusive of leakage.

DESIGN OF COFFERDAM

No precedents were found for an independent cofferdam so large and high. Those most nearly comparable with it were constructed for the raising of the battleship "Maine" in Havana Harbor and at Black Rock, Buffalo, N. Y., both of which were of the filled-pocket type and were designed to be independently stable. Extended analyses and computations showed that pockets large enough to possess independent stability due to the width of base and the gravity moment would require very large dimension and that the material available for filling was not suitable or satisfactory and could not be depended upon to prevent serious distortion and probable displacement, such as occurred in the Havana cofferdam. It was believed that a single line of steel sheeting to serve as a water-tight diaphragm or core for a dam embankment would not have sufficient stability in the soft river bottom before it was reinforced by the embankment, and it was therefore determined to provide pockets large enough to give independent stability during construction, but not large enough to resist the final working pressure, and to reinforce them by the deposition of heavy fills of riprap in the interior of the dam which are calculated to be strong enough to resist the thrust, while the pockets filled with soft material exclude the water. Under the maximum unbalanced pressure head of 68 ft. the horizontal tension developed in the steel-sheet piles is calculated at 4640 lb. per linear inch throughout the pocket, which, together with the silting effect of the fill, is relied upon to provide tightness. The strength of the joints between the sheet piles is about 9000 lb. per linear inch, which considerably exceeds the upper limit within which the pressure can be maintained during construction. The cofferdam pockets, with convex inner and outer faces from 16 to 22



TEMPLATES SET FOR DRIVING SHEET PILES



COFFERDAM POCKET READY FOR FILLING

ft. apart, and straight parallel ends 24 ft. apart, were adopted instead of circular pockets because they were less liable to be distorted and gave a much greater minimum thickness.

The river side and downstream end of the cofferdam are composed of sheet-pile pockets filled with dredged material and reinforced on the inside with a heavy bank of riprap. At the upstream end of the cofferdam, where the height is not so great, the pockets are replaced by a single line of sheet piles reinforced by inside riprap and additionally secured by horizontal steel anchor rods 3 in. in diameter, laid above low-water level in trenches perpendicular to the line of piling. One end of each rod has a nut and washer bearing on the ranger inside the cofferdam and the other end has a similar bearing on a chock across the faces of two anchor piles driven in the adjacent crib pier.

At the site of the cofferdam the bottom was dredged to 30 ft. below water level, leaving a mud bottom to support the sheet piles and form a seal on top of the rock, and a portion of the interior was dredged to the final depth of 44 ft. or to the surface of the rock.

CONSTRUCTION OF THE COFFERDAM

All sheet piles are 37 lb. Lackawanna units in pieces of a maximum length of 73 ft., penetration of 34 ft., and projection of 9 ft. above low-water level. The sheet piles in the single lines are assembled against the face of a single exterior ranger, and those in the pockets are assembled around the exterior of a wooden template fastened to four wooden piles driven for the purpose. They are assembled by the 102-ft. spliced and trussed wooden boom of a floating derrick and usually settle under their own weight alone to within about 10 ft. of the rock surface. They are then driven in sets of two or three to a moderate refusal, after which they are driven singly to their final refusal by Arnott and New Monarch steam hammers.

Work is now in progress with a force of about seventy-five men and it is expected that the cofferdam will be unwatered this season and that the rock excavation will be carried on during the winter. The pier was designed and the work is being conducted under the direction of the Department of Docks, of which R. A. C. Smith is commissioner and C. W. Staniford chief engineer.

WATER FOR CORDOBA, Argentina, is to be brought 28 mi., from a point on the San Roque River 1257 ft. above the city, according to a recent consular report. The estimated cost, including filters and a tunnel 5781 ft. long, is $4,600,000.

Break Interrupts Irrigation

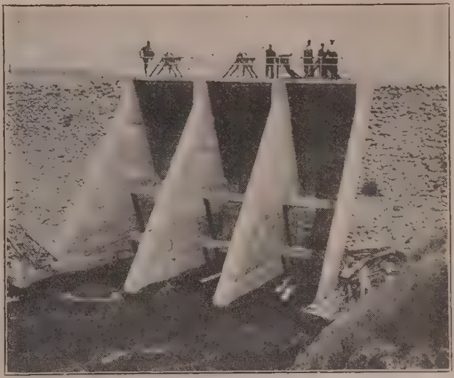
Failure of Reservoir-Gate Wing Wall Caused by Seepage at Turlock, California

DESTRUCTION of more than a mile of main canal in the Turlock irrigation district, California, followed the break, June 27, in the west wing wall of the outlet gate of a newly completed regulating reservoir. The Turlock irrigation district is situated 126 miles southeast of San Francisco, just above the junction of the Tuolumne and San Joaquin Rivers. It takes its water supply from the former stream and serves 175,000 acres through a 23-mile main canal and a system of laterals totaling 200 miles in length. This season a newly completed regulating reservoir with a capacity of 48,000 acre-feet was put in service and when filled for the first time the west wing wall of the outlet gate gave way, opening a gap in the embankment about 60 ft. wide and releasing the stored water. A telegraphic report of the break was printed in the Current News section of the Engineering Record of July 4.

The intake of the system is at the La Grange dam on the Tuolumne River. One-third the flow at the dam is accorded the Modoc irrigation district with a flume on the east side of the stream, while the Turlock district is entitled to two-thirds of the flow and has its intake on the west bank. Early last year construction was started on a reservoir in the Turlock district about 10 miles below the dam. This was intended chiefly for regulating purposes and was formed by supplementing a natural basin with sixteen small earthfill dams and the outlet gate. The canal formerly passed directly across the reservoir site, and 3 miles of its length were submerged in the 32,000 acres of flooded area.

THE OUTLET GATE

The outlet gate is a hollow-type reinforced-concrete structure with three 8 x 10-ft. sluices closed by steel Tainter gates. The height from sill to operating platform is 40 ft. 8 in. and the length between wing walls is 38 ft. When examined by a member of the Engineering Record's editorial staff on July 8 practically no damage to the main structure could be detected, the failure being confined to the west wing and embankment, a section of which had been swept out down to a level about 40 ft. below the crest. The peculiar feature of the break was that the cutoff wall had not been undermined at or near the main structure, so far as could be determined, but seepage beneath the toe of the 6-in. concrete facing on the earthfill had apparently removed the support behind the concrete wing wall, which collapsed at its weakest point. Then the outrush of waters completed its destruction.


DOWNSTREAM SIDE OF OUTLET GATE

When the gate was built the cutoff wall was carried down 6 ft. in "hardpan" below the sill level, which in turn is 4 ft. below the level of the original canal bottom. This depth was all that was considered necessary, and it is now generally agreed that the material at the site classed as hardpan has proved to be less stable than was expected. Now that the formation is fully exposed it is evident that considerable difference exists in the several horizontal strata, some of which contain seams of poor material.

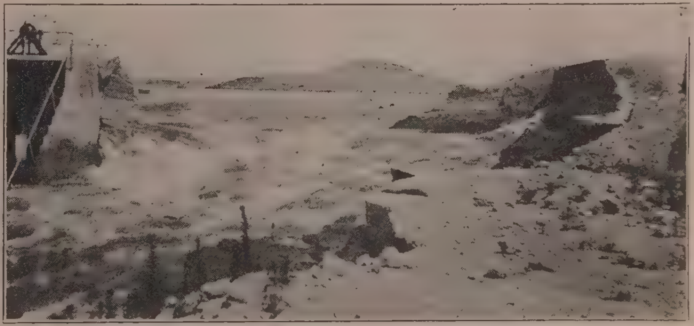
Such conclusions as can be drawn so early by engineers of the district indicate that in repairing the structure special attention should be given to placing a heavy, impervious paving on the earthfill and, if the hollow-type gate structure is extended to close the breach, making a continuous cut-off deeper than before. A concrete apron edged with steel-sheet piling along the upstream side is being considered.

EFFECTS OF THE BREAK

When the break occurred the canal below the gate filled and overflowed at numerous points for a distance of 2 mi. or more. At a point of about 1½ mi. below the reservoir a main opening developed and quickly cut to a level below the canal bottom. This became the vent through which the entire volume of stored water passed, and observations of reservoir level made at the time of the break indicate that in the first hour the level lowered 1 ft., which is the equivalent of an average rate of flow for the hour of 36,000 sec.-ft. This flow passed into an unlined earth canal, originally 60 ft. wide on the bottom and designed for a maximum capacity of 1270 sec.-ft.

At the point where the flow left the canal the Tuolumne River passes near at a much lower level and with no high ground intervening. The break in the canal was cut about 75 ft. deep and 250 ft. wide before the flow stopped. As the geological formation is a friable hardpan, an irregular gorge of varying depth was cut by the cur-

UPSTREAM SIDE OF OUTLET, RESERVOIR NEARLY EMPTY


VIEW THROUGH BREAK BESIDE OUTLET GATE

rent along the route of the old canal channel, which was entirely destroyed. This gorge continued nearly 200 ft. wide for about ½ mi. above the vent.

Reconstruction

The part of the main canal destroyed had given trouble for some time due to its proximity to the lower level of the river bed, which made slight leaks dangerous, and in fact an alternative route for the first 3 mi. of this main had been surveyed some time ago. The recent break, therefore, simply hastened the construction of the new canal, and in order to get water back in the ditches at the earliest moment possible a plan was adopted whereby only 1½ mi. could be by-passed around the break. One mile of this emergency construction follows the route of the permanent 3-mi. alternative line behind a hill from the river bed, while the other ½ mi. represents a temporary short cut to intercept the old canal a short distance below the washout.

The emergency work calls for the moving of about 100,000 cu. yd. of material, of which 80,000 cu. yd. will be on permanent construction. A few days after the break about 200 men and 500 horses were at work on the alternative canal, and, barring the possibility of striking material which cannot be handled by Fresno scrapers in the one heavy cut, which is 500 ft. long and has a maximum depth of 16 ft., the water is expected in the ditches again by Aug. 1.

Meantime reconstruction at the reservoir is in progress, and it is planned to complete the repair to such a level by Aug. 1 that water can be passed through the gate into the old canal channel which is to be used for a short distance. The flow into the reservoir has been stopped and, under the protection of cofferdams, the gate foundation is being unwatered. Final decision on the details of repair will not be made until an examination of the foundations can be made to confirm or refute present conclusions.

E. Kiernan is president of the Turlock irrigation district and the emergency construction is being carried out under the direction of R. V. Meikle, chief engineer.

Preserving an Ancient Roof

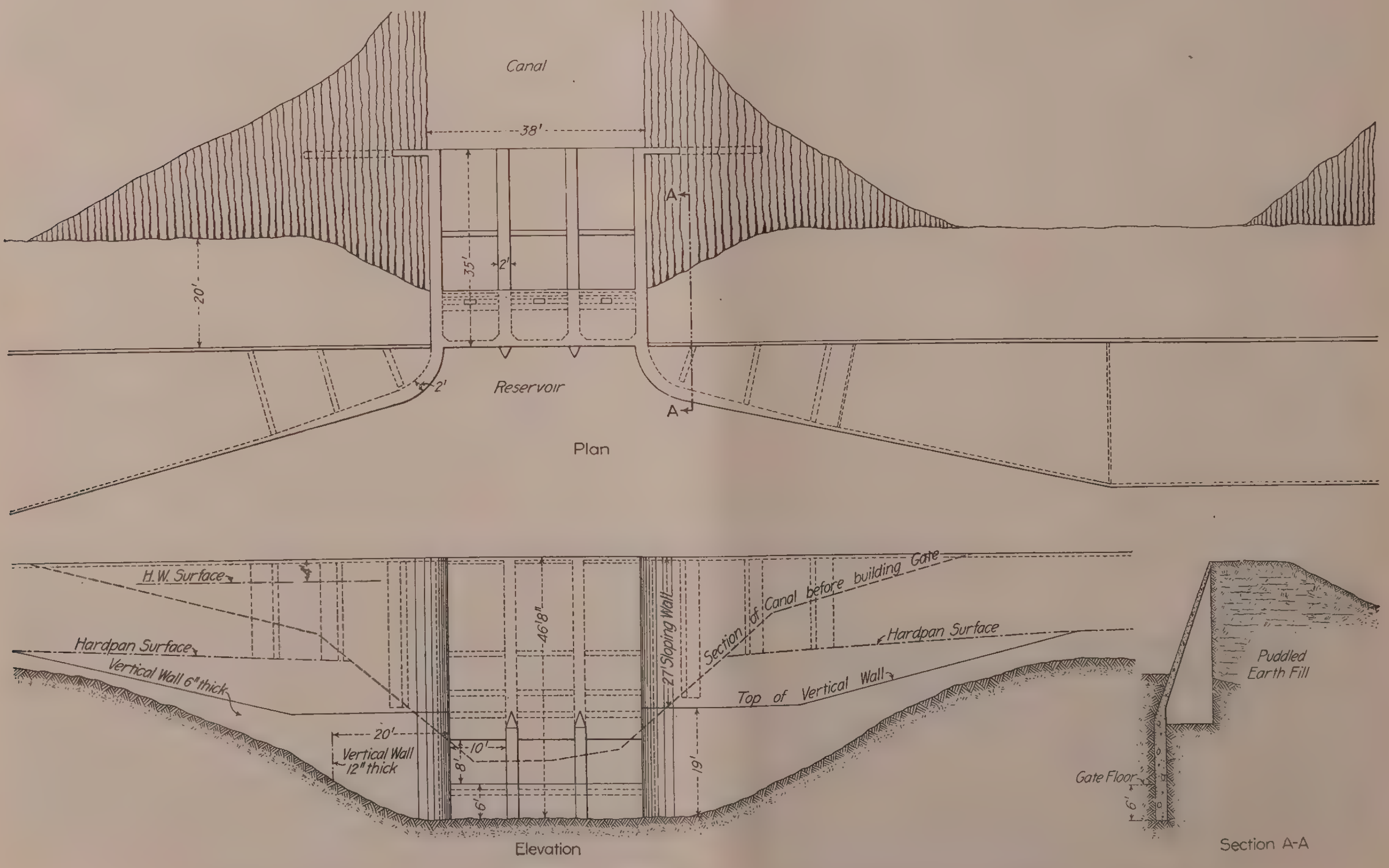
Costly System of Steel Reinforcement to Support Decayed Long-Span Timber Trusses of Great Historical Interest

The roof of Westminster Hall, London, is supported on thirteen oak trusses said to be the earliest and best-known example of the hammer-beam type. The roof is Gothic and consists of a combination of cantilever and arch members of 64½-ft. clear span, with vertical posts supporting steeply inclined rafters. The trusses are seated on heavily buttressed masonry walls, and according to an article in "Engineering" of London have been in service more than 500 years without receiving other repairs than those required by the decay of the material. The trusses are of a complicated design, with intermediate stresses, and have been reinforced with modern timber struts and tiebolts.

A recent investigation showed that some of the timber was so rotten and disintegrated that large pieces could be picked out, and that the ravages of insects, particularly a certain species of beetle, were much more serious than dry rot and surface decay. At least four of the thirteen trusses were in a dangerous condition and likely to fall.

A plan submitted by Architect F. Bains for the preservation of the roof at an estimated cost of about $300,000 has been accepted. The old trusses will be reinforced on the lines of their principal members by nondescript steel frameworks, which will be entirely concealed. The original appearance of the building will be restored and maintained and all large cavities in the timber will be pieced up with new oak. The trusses are in such a serious condition that they must not be lifted or jacked in any way, but must be maintained absolutely in their present positions, without wedging or drawing together. Some of the joints are so decayed that the roof would be likely to collapse if any attempt were made to lift and carry it by the trusses. In order to accomplish the gradual removal and reconstruction, the work will be carried on from a movable steel scaffold, weighing 115 tons, that will support from two to four trusses simultaneously.

FOREST-FIRE PROTECTION in the Angeles National forest in Southern California has been supplemented this year by the first of a series of lookout towers to be erected at strategic points. The first tower is 100 ft. high and is being erected on Sunset Peak between San Antonio and San Dimas Canyons. The tower will be surmounted by an inclosed platform large enough for the observer's bed as well as the equipment, and the observer is to be on guard continually. A high-power telescope will be provided, by means of which officials are to be notified by telephone, in addition to the first alarm, of the direction of the wind, the progress of the fire and other detailed information. From this tower it is expected that any fire within 50 mi. can be accurately located.



PLAN AND ELEVATION OF OUTLET GATE AT RESERVOIR OF TURLOCK IRRIGATION DISTRICT, CALIFORNIA

Spacing of Piers for Reinforced-Concrete Pipe

Method of Calculation, Taking into Consideration Stresses Due to Water Pressure, Fill and Span

By RICHARD MULLER
Hydraulic and Electrical Engineer

REINFORCED-CONCRETE pipe consists in the incorporation of a steel structure composed of circular and longitudinal steel bars embedded in the concrete. These reinforcing bars are calculated to carry the stresses due to water pressure, fill and span, and the concrete is simply considered as a filling material.

In the case of a reinforced-concrete pipe supported on piers the upper part of the pipe, or that located above the neutral axis, may be allowed to carry compressive stresses. The steel area may be calculated by comparing the circular reinforcement to an equivalent full steel shell, then numbering and sizing the circular bands in such a way that the equivalent area of metal is obtained. The longitudinal bars take care of temperature stresses. A pipe may also perform the function of a beam in which compressive but not tensile stresses are allowed.

METHOD OF CALCULATION

The following method for calculating the proper relations in the problem of pier spacing is derived by M. P. Caufourier: Assume the steel reinforcement to be composed of circular and longitudinal bars. The circular bars are calculated to resist the water pressure and pressures that may cause deformations, such as those due to fill. The longitudinal reinforcement, however, must take care of the bending moment between supports. These longitudinal bars, uniformly spaced, will be replaced by a fictitious pipe, the steel shell of which shall have an equivalent area.

In the figure let r = mean radius of pipe; e, thickness of concrete shell; ε, thickness of shell of equivalent fictitious pipe; NN', neutral axis defined by angle α; α, angle limited by radius ON on the plane of bending, and n, ratio of coefficients of elasticity of concrete and steel.

Considering that tension does not exist in concrete, e does not have to be considered below the neutral axis. The position of the neutral axis is determined by the expression

$$\tan \alpha - \alpha = n\pi(\varepsilon/e) \qquad (1)$$

which may be resolved by means of tables of arc lengths and natural functions of tangents.

The moment of inertia with respect to the neutral axis just determined is

$$I = 2r^3 n\varepsilon\,(\pi/2 + \pi\cos^2\alpha) + 2r^3 e \times (\alpha/2 - \tfrac{3}{4}\sin 2\alpha + \alpha\cos^2\alpha) \qquad (2)$$

The maximum stresses are then found by the formulæ

$$R_{tension} = nM_1 r\,(1 + \cos\alpha)/I \qquad (3)$$

$$R_{compression} = nM_1 r\,(1 - \cos\alpha)/I \qquad (4)$$

$$R_{concrete} = M_1 r\,(1 - \cos\alpha)/I$$

in which M_1 is the maximum bending moment.

From these formulæ the following relation is found:

$$\frac{R_{concrete}}{R_{tension}} = \frac{1 - \cos\alpha}{n\,(1 + \cos\alpha)}$$

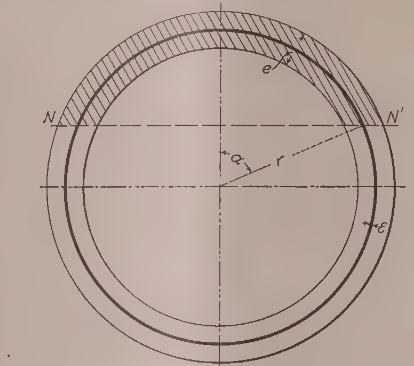
This gives α when $R_{tension}$ and $R_{concrete}$ are known. The value of α substituted in equation (1) gives the thickness ε, which is the unknown quantity of the problem.

Suppose it is required to design a reinforced-concrete pipe of 5-ft. inside diameter to resist a maximum water pressure (including effect of water hammer) of 50 ft. The pipe will be laid in wet sandy soil and covered with a 4-ft. fill. There will be two layers of reinforcement, one near the outer and the other near the inner face of the pipe. It will be assumed that the thickness of the concrete shell is 6 in.

If the steel alone must resist the water pressure, and the limiting stress is 12,500 lb. per square inch, the shell thickness of an equivalent steel pipe will be

$$\varepsilon = 2.6\,(50 \times 5/12{,}500) = 0.052 \text{ in.}$$

using a coefficient of joint efficiency of 100 per cent. The thickness as determined corresponds to 0.63 sq. in. of steel per linear foot of pipe.



STRESSES DUE TO FILL

Fill is considered as being uniform and of an average depth of 6 ft. (For a full discussion of this subject, see Bulletin 22 of the University of Illinois, "Tests of Cast-Iron and Reinforced-Concrete Culvert Pipe," by A. N. Talbot.) If w_v and w_h are respectively intensity of vertical and horizontal forces, and if the intensity of the horizontal pressure is taken as one-third of the vertical and the approximate depth to the center line 6.66 ft., $w_v = 100 \times 6 = 600$ lb. and $w_h = 100 \times 6.66/3 = 222$ lb. per square foot. Substituting in the well known formula

$$M_b = (1 - w_h/w_v)\, w\, (r^2/4)$$

it is found that

$$M_b = (1 - 222/600)\ 600\ (2.5^2/4) = 590.63 \text{ ft.-lb.} = 7087.5 \text{ in.-lb.}$$

The area of reinforcement will be determined by

$$M = sq \times 0.86d$$

where s is the unit stress of steel and $q = M/s \times 0.86d$. But $d = 5$ in., it being assumed that the distance from center of steel to face of concrete is 1 in. Therefore

$$q = 7087.5/(12{,}500 \times 0.86 \times 5) = 0.132 \text{ sq. in.}$$

This amount of steel is required along both faces. Having determined the amount of steel that will resist the water pressure and the amount that will resist the stresses due to fill, the total required amount will be $0.63 + (2 \times 0.132) = 0.894$ sq. in. It will be necessary to use ⅝-in. round bars spaced 4 in. center to center.

As this same pipe is supposed to cross a declivity of the ground, and be supported by piers, it is necessary to apply the formulæ developed by Caufourier for determining the proper spacing. The thickness, ε, of an equivalent fictitious shell is 0.052 in. (It is assumed that the longitudinal reinforcement is of the same area as the transverse reinforcement.) The thickness, e, of the concrete shell is 6 in. The position of the neutral axis will be determined by proper substitution in formula (1). It is found that the value of α that satisfies the equation is approximately 50 deg. 20 min., for which the arc value is 0.878 and $\tan\alpha$ is 1.2.

In the formula

$$\frac{R_{concrete}}{R_{tension}} = \frac{1 - \cos\alpha}{n\,(1 + \cos\alpha)}$$

let $R_{tension} = 12{,}500$ lb. per square inch, $n = 29{,}000{,}000/2{,}400{,}000 = 12$, $\cos\alpha = 0.638$. Solving for $R_{concrete}$, a value is found of approximately 230 lb. per square inch. Remembering that $\cos^2\alpha = 0.407$ and that $\sin 2\alpha = 0.185$, the moment of inertia will be determined by formula (2),

$$I = 2 \times 12 \times (2.75 \times 12)^3 \times 0.052\,[\tfrac{1}{2}\pi + \pi \times 0.407] + 2 \times (2.75 \times 12)^3 \times 6\,[0.878/2 - \tfrac{3}{4} \times 0.185 + 0.244 \times 0.407] = 688{,}400 \text{ in.}^4$$

in round figures.

It is assumed that the pipe is acted upon as a uniformly loaded beam and that expansion joints are provided at each support. The maximum bending moment is given by $M_1 = Wl/8$ and the value of W is taken as 2500 lb. per linear foot (weight of pipe shell and water within). Then $M_1 = 3750l^2$ in.-lb., l being in feet. Substituting this value in formula (3) and solving for l,

$$12{,}500 = [12 \times 3750 \times l^2 \times 33(1 + 0.638)]/688{,}400,$$

and

$$l = 59.5 \text{ (say 60) ft.}$$

Knowing the safe span, the compressive stress for steel will be found by proper substitution in formula (4). In the present case compression is very small, taking into account the considerable thickness of concrete. $R_{compression}$ is found to be 2760 lb. per square inch.

Finishing a Segmental Girder

THE San Gabriel River bascule bridge near Los Angeles has a 187-ft. Scherzer rolling-lift bascule span with double-web segmental girders weighing 55,000 lb. each. The flanges curve through an arc of nearly 90 deg. with a radius of more than 30 ft. and were planed by an ordinary machine-shop lathe with its tool-stock connected to the girder by a cable. The girder was laid flat on a pair of curved rails and rigidly attached by timbers to a pivot in a small concrete pier at the center of the curve. The cable was arranged so that the feed of the lathe would pull the girder in a circle around the center of the curve. By means of its adjustment and a belt from a motor over a large pulley the lathe was given a very slow speed and pulled the girder against the tool of a milling machine, which accurately finished the curved surface. The work was supervised by Samuel Murray, bridge engineer of the Oregon-Washington Railroad & Navigation Company, and was described by him in a paper on "Some Types and Features of Movable Bridges," presented to the Pacific Northwest Society of Engineers.

531-FOOT CHANNEL SPANS OF THE NORTH SIDE POINT BRIDGE

North Side Point Bridge, Pittsburgh

Two 531-Foot Spans with Reinforced-Concrete Pier Caissons Sunk Partly by Dredging Through Open Wells and Partly Under Pneumatic Pressure

THE North Side Point highway bridge across the Allegheny River, Pittsburgh, Pa., will, it is estimated, cost $940,000. It has two 531-ft. steel truss river spans 60 ft. wide. Their pier foundations were built with reinforced-concrete caissons sunk mainly by dredging through open wells, although the final excavation was made after the wells were closed and the working chambers were put under pneumatic pressure and the surface of the rock prepared by hand. The high shore piers were designed to receive unbalanced arch thrusts. The center pier, in midchannel, was designed to resist flood and ice pressures and was constructed on an artificial island formed by building a cofferdam around the site and filling it with material dredged from the river bottom. The weight of the superstructure is distributed on the piers by grillages embedded in the concrete. The bottom lateral system, the cantilever sidewalk brackets and the expansion shoes have unusual features.

PIERS

Each of the two shore piers is located on dry land and was designed to carry one end of a river span and on the shore side to take the reaction from an 85-ft. concrete arch approach span. The concrete arch span will rest on a horizontal offset made in the pier just above its belt course and about 32½ ft. below the bridge seat for the river span.

In the concrete at the top of the piers there is embedded a grillage, two beams of which, spaced 4 ft. apart, are nearly as long as the pier and connect the groups of I-beams 14 ft. long under each shoe of the river spans. All beams were assembled together with tie rods and pipe separators, and on either side of the span shoe a heavy angle was securely bolted to the top of each beam to bring their tops to a uniform height.

The river pier was designed to take one end of each of the steel spans and to be stable under a horizontal side pressure of 42,000 lb. per linear foot applied about 60 ft. above its base, to provide for a possible ice gorge on one side during high water before the spans were erected, as it was expected that the pier would stand in the river through one winter before the steel spans were erected.

The pier is symmetrical about its longitudinal and transverse center lines. In the concrete at the top of the pier there is embedded a plate girder grillage extending to within 2 ft. of the end of the pier. The outside girders and the end plates were provided with two tiers of shelf angles, with holes for anchors for securing the stone facing. On the girder grillage there is placed under the shoes of each truss a transverse grillage composed of I-beams riveted together through vertical end plates, riveted to the tops of the girders, and filled in with concrete to provide a bed for the expansion ends of the trusses.

STEEL SPANS

The two duplicate spans have a pair of pin-connected modified Baltimore trusses with subdivided panels. They are 92 ft. 10½ in. deep at the center and 58 ft. 1 15-16 in. deep at the hip, and are spaced 40 ft. apart on centers. They carry a roadway 36 ft. wide in the clear between curbs, provisions being made for two street-railway tracks near the center of the roadway. There are two sidewalks, having a width of 12 ft. from curb to outer face of fascia girder, carried by cantilevered brackets.

The floor system, hangers and sub-panel struts were proportioned for a live load consisting of a continuous line of 52-ton, double-track cars 45 ft. long on each track and a concentrated load of 20 tons on two axles 10 ft. apart on each side of track, plus a uniformly distributed load of 100 lb. per square foot on the remainder of the roadway and the entire sidewalk floors.

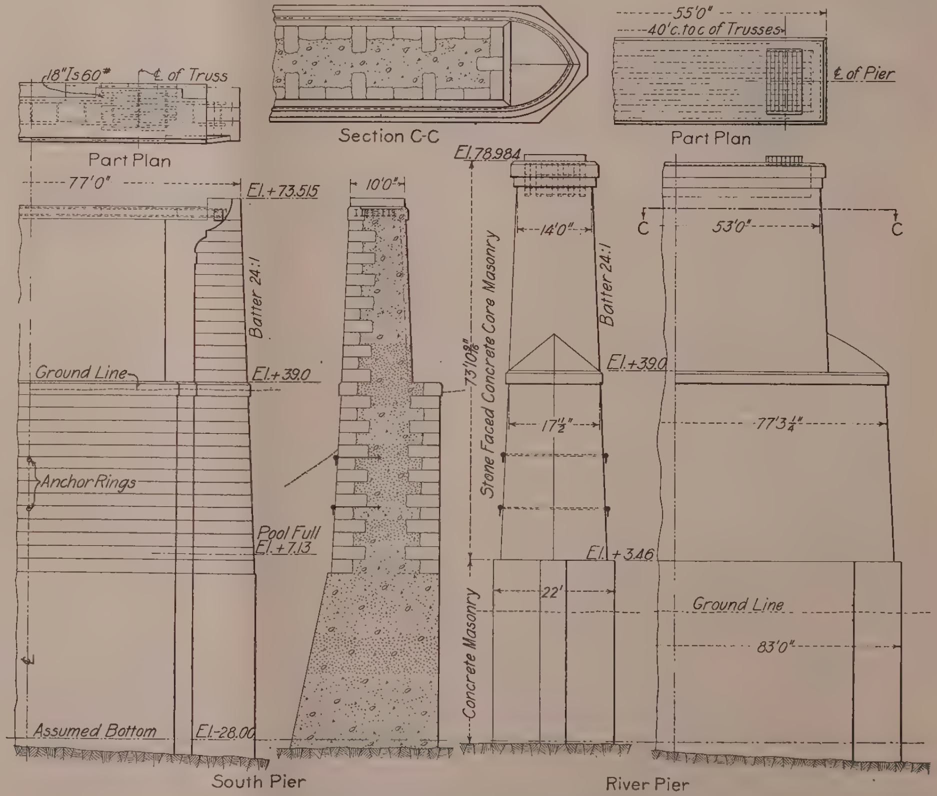
The trusses were proportioned for a live load of 5336 lb. per linear foot of bridge. In addition to the dead and live load stresses there was added an impact increment, determined by the formula $I = S \times 150/(L + 300)$ where I is impact to be added, S is maximum live load stress, and L is length in feet loaded to produce maximum live-load stress.

The lateral systems were proportioned for a wind load of 30 lb. per square foot on exposed surfaces of both trusses and the floor system as seen in elevation, wind load being treated as moving load.

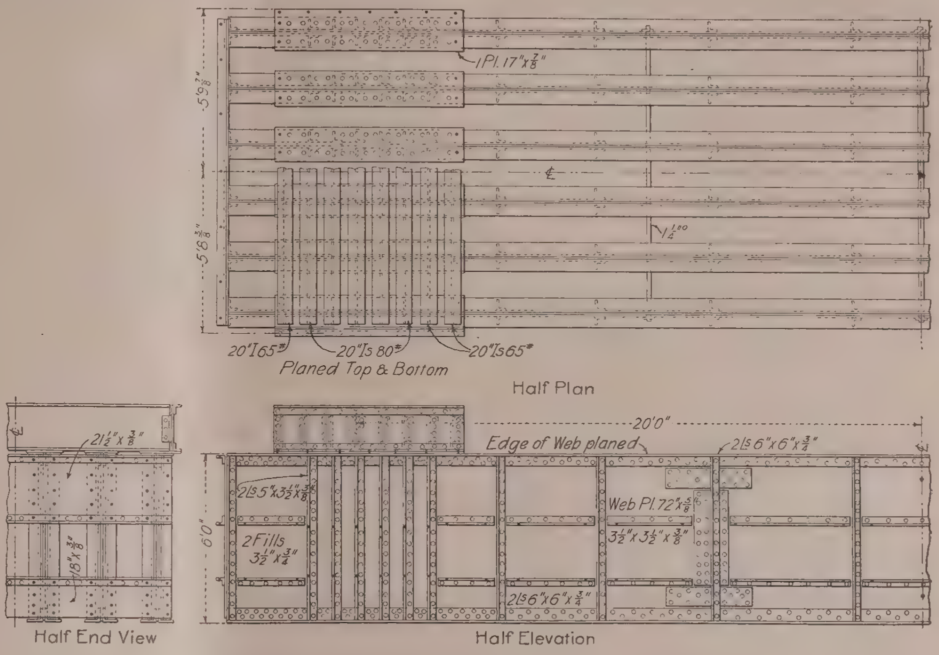
PRINCIPAL MEMBERS

The design of the steelwork conforms in general to that of high-class railroad-bridge construction. Instead of transmitting the top lateral stress through the inclined end posts of the trusses it is carried through the false top-chord panels and transmitted to the shoes through the vertical end posts, between which there is a heavy box plate-girder portal strut.

The bottom lateral diagonals are single angles riveted to the bottom flanges of the stringers and to connection plates which are connected to the floor beams and bottom chord pins of the trusses. These later-



GENERAL DESIGN OF PIERS FOR MAIN SPANS



GRILLAGE AND DISTRIBUTING GIRDERS ON RIVER PIER

als were not required after erection, as the buckle-plate floor on the roadway provides ample lateral bracing. The function of the bottom laterals was to provide means of lining up the trusses and provide lateral bracing during erection.

The sidewalk cantilever brackets, which are a little more than 9 ft. long, are of very heavy construction, as they were designed to support, in addition to the dead and live loads already mentioned, the weight of water and gas pipes and electric-wire conduits, amounting to 800 lb. per linear foot of walk. These brackets are field-riveted to the vertical truss posts through hitch angles and have their top members, which are composed of two channels, connected to the top flanges of the main floorbeams by tension yoke plates passing around the posts.

Both spans have expansion bearings on the river pier. The expansion shoes rest on segmental rollers set in closed boxes filled with vaseline.

The total weight of the two steel spans, including hand railing, is 4947 tons and cost $298,702.00. Four bids were received for the steel spans, ranging from $297,792.00 to $253,000.00, covering that part of work covered by lump-sum item, and the work was awarded to the American Bridge Company, which was the lowest bidder.

MAIN-SPAN FALSEWORK AND TRAVELER

SINKING PIER FOUNDATIONS

The site of the center pier was inclosed by a cofferdam of steel-sheet piling driven in water about 20 ft. deep; the cofferdam was then filled with gravel taken from the river in the vicinity of the pier, thus forming an island. On this island were constructed the forms for the bottom section of the concrete caisson having a steel cutting edge and reinforcement of vertical, horizontal, and inclined rods. The caisson was provided with a working chamber, and above this chamber the caisson was solid, except for four vertical wells 6 ft. in diameter with their lower ends belled out to the full width of the top part of the working chamber.

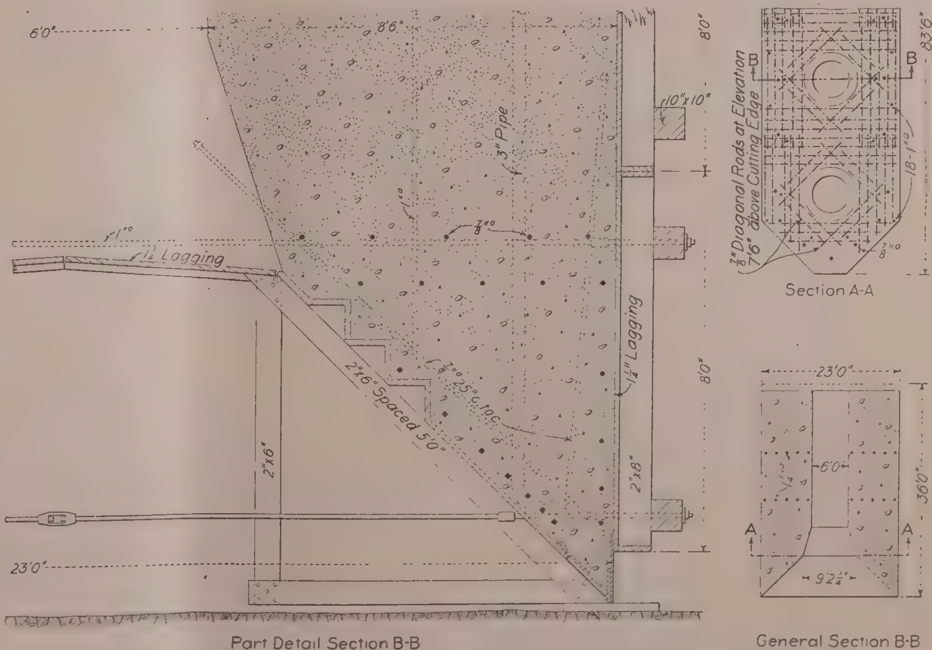
After the forms for the walls and roof of the working chamber were constructed, the space under them, above the surface of the island, was filled solid with sand and gravel, tightly packed to support the forms and heavy mass of wet concrete of the caisson. The nails through the tops of the inclined pieces supporting the walls of the forms were removed after the sand and gravel cover was placed, to prevent the timbers from developing arch or truss action when the forms were loaded, thus eliminating one difficulty in removing forms.

The caisson was sunk by dredging through the open wells with orange-peel buckets operated by derrick boat, care being taken to dredge uniformly in all the wells, the buckets being shifted from well to well frequently and thus causing the caisson to settle evenly.

As the caisson sunk, 8-ft. sections were added until the full height of the concrete base of the pier was constructed. Each section was reinforced with horizontal longitudinal rods near its top. The cutting edges for all piers were shaped to correspond with the surface of the rock as indicated by the city's test borings. Twenty-six 3-in. pipes were built into the concrete caisson, with their open lower ends projecting below the concrete a short distance inside the cutting edge, so that they could be used for hydraulic jets to scour the bottom and assist in sinking the caisson. They were not, however, required for this purpose, and so served only as additional reinforcement to the concrete. After sinking about 20 ft. by the dredging process the tops of the wells were closed by air locks, and the sinking was continued to bedrock under pneumatic pressure.

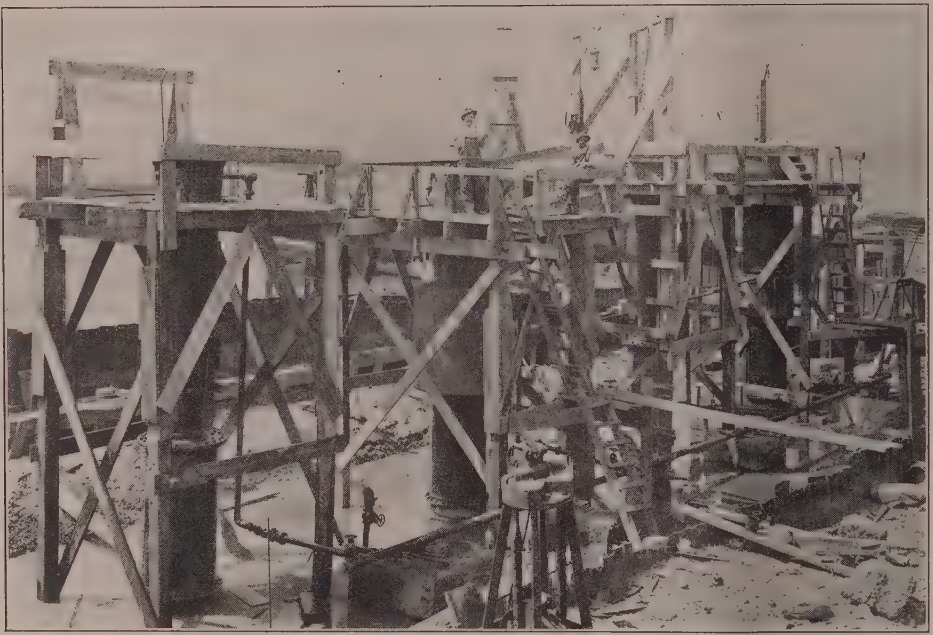
The entire work was carried on in a manner similar to that used by the same contractor for the piers of the Beaver Bridge, which were illustrated and described in the Engineering Record of Jan. 28, 1911, page 92.

The total cost of the three piers was $206,914.60. Eight bids were received for the three piers ranging from $182,750 to $206,000 for that part of work covered by lump-sum item, and the work was awarded



DIAGRAMS AND DETAIL OF WORKING CHAMBER AND FORM FOR RIVER PIER to the Dravo Contracting Company, the lowest bidder.

The work was designed and executed under the direction of the bureau of construction, Department of Public Works, of which N. S. Sprague is superintendent, Emil Swensson, consulting engineer, and T. J. Wilkerson, division engineer in charge of the designing and construction.



AIR LOCKS FOR CENTER-PIER CAISSON, NORTH SIDE POINT BRIDGE, PITTSBURGH

Machine for Testing Columns Subjected to High Temperature

THE Committee on Fireproofing Materials of the American Society for Testing Materials reported at the convention in Atlantic City that the Underwriters' Laboratories, at Chicago, are now erecting a machine for testing fireproofed steel columns when subjected to high temperature. The work is being done in cooperation with the Factory Mutual Fire Insurance Companies, while the United States Bureau of Standards has been enlisted to assist in the temperature measurements. The heavy foundations and framework have been erected, but it will probably be some months before the apparatus and furnace are completed and in shape for actual tests. It has been designed for a wide range of tests and accurate control of temperature.

"There is a special demand," the report says, "for a series of tests of heat-retardant coverings of moderate cost for columns and girders of isolated factories, which, while not of the highest standard of efficiency, could still be relied upon to give ample time for escape of operatives and for bringing fire extinguishing apparatus into use. Considerable material for these tests has been already secured."

Until the actual testing has at least been started the committee believes it unwise to outline a test that may have to be modified when more experience is gained.

WATER DEPARTMENT motor trucks in Milwaukee cost $100 less for upkeep than a horse and wagon, with the efficiency about double, according to the 1913 report of the water department. Much of the work is on the city's outskirts.

Statistics of Steam Railways in the United States

Figures from Preliminary Abstract of Annual Report of Interstate Commerce Commission for Year Ended June 30, 1913

FOLLOWING the usual custom, the Interstate Commerce Commission has issued an abstract of statistics of the steam railways in the United States, based upon compilations for its twenty-sixth annual statistical report, covering the fiscal year ended June 30, 1913. The figures may be somewhat modified in the full report, as they do not include the returns for switching and terminal companies, nor for carriers having gross operating revenues for the year of less than $100,000.

Mileage.—The roads covered by this abstract represented 244,418.49 mi. of line operated, including 11,162.97 mi. used under trackage rights. The aggregate mileage of railway tracks of all kinds covered by operating returns for these roads was 369,579.80 mi. This mileage was thus classified: Single track, 244,418.49 mi.; second track, 26,270.55; third track, 2,588.68; fourth, fifth and sixth tracks, 1,964.06; yard track and sidings, 94,338.02. These figures indicate an increase of 8,628.36 mi. over corresponding returns for 1912 in the aggregate length of all tracks, of which increase 3,157.59 mi., or 36.59 per cent, represent yard track and sidings.

Equipment.—There were 63,378 locomotives in their service on June 30, 1913, an increase of 2102 over the previous year. Of these, 14,396 were passenger, 37,924 freight, 9834 switching, and 1224 were unclassified.

The total number of cars was 2,445,508, or 76,566 more than on June 30, 1912. This equipment was thus assigned: Passenger service, 51,700 cars; freight service, 2,273,564; company's service, 120,244.

Of cars in freight service, there were classified 2,273,289, as follows:

NUMBER OF FREIGHT CARS IN SERVICE

Description	Number	Aggregate capacity, tons
Box	1,032,585	35,607,134
Flat	147,541	5,151,054
Stock	78,308	2,421,827
Coal	871,339	38,314,920
Tank	8,216	327,727
Refrigerator	43,389	1,357,403
Other cars in freight service	91,911	3,798,080
Total	2,273,289	86,978,145

The average number of locomotives per 1000 mi. of line was 259, and the average number of cars, 10,005. The number of passenger-miles per passenger locomotive was 2,341,269, and the number of ton-miles per freight locomotive was 7,843,663.

Employees.—The total number of persons on the payrolls of these roads on June 30, 1913, was 1,815,239, or an average of 743 per 100 mi. of line. As compared with corresponding returns for June 30, 1912, there was an increase of 115,298. There were 67,026 enginemen, 70,477 firemen, 52,086 conductors, 146,855 other trainmen and 38,253 switch tenders, crossing tenders and watchmen.

The complete report will include summaries showing the average daily compensation of eighteen classes of employees for a series of years, and also the aggregate amount of compensation reported for each of the several classes. The total amount of wages and salaries reported as paid to railway employees during the year ended June 30, 1913, for the roads under consideration was $1,373,830,589.

Capitalization.—On June 30, 1913, the par value of the amount of railway capital outstanding was $19,796,125,712. This amount includes capital held by the railway companies concerned, as well as by the public. Of the total amount of such capital outstanding there existed as stock $8,610,611,327, of which $7,231,515,045 was common and $1,379,096,282 was preferred; the remaining part, $11,185,514,385, representing funded debt, consisted of mortgage bonds, $8,186,366,426; collateral trust bonds, $1,189,636,796; plain bonds, debentures and notes, $1,107,076,783; income bonds, $250,290,655; miscellaneous funded

EXPENDITURES FOR ADDITIONS AND BETTERMENTS AND FOR NEW LINES AND EXTENSIONS

	Expenditures for additions and betterments	Expenditures for new lines and extensions		
Investment to June 30, 1913			$16,351,639,266	
Investment to June 30, 1912			15,874,579,626	
Increase 1913 over 1912				$477,059,640
From cash or other working assets	$329,511,772	$54,819,241		
From special appropriations	48,079,165	473,281		
Through issue of securities	162,597,278	63,692,746		
Not assigned to any of the above classes	5,801,127	10,263,251		
Total	$545,989,342	$129,248,519	$675,237,861	
Miscellaneous charges not classified			4,847,012	
Total expenditures during year			$680,084,873	
		Credits		
Property retired or converted		$79,495,571		
Adjustments		5,058,790		
Difference between record value of grantor and purchase price of grantee in cases of roads sold, merged, consolidated, etc.		118,470,872		
Total			$203,025,233	
Net increase during year				$477,059,640

obligations, $82,858,275, and equipment trust obligations, $369,285,450.

Of the total capital stock outstanding for the roads under consideration, $2,836,023,744, or 32.94 per cent, paid no dividends. The amount of dividends declared during the year (by both operating and nonoperating companies represented) was $368,606,327, being equivalent to 6.38 per cent on dividend-paying stock. The average rate of dividends paid on all stocks outstanding pertaining to the roads under consideration was 4.28 per cent. No interest was paid on $1,128,776,748, or 10.44 per cent, of the total amount of funded debt outstanding (other than equipment trust obligations).

Investment.—The expenditures for additions and betterments, as well as the expenditures for new lines and extensions, during the fiscal year 1913 are analyzed in the tabular statement at the bottom of the preceding page.

SERVICE, REVENUES AND EXPENSES

Public Service.—The number of passengers carried during the year was 1,033,679,680, an increase over the previous year of 39,307,397. The number of passengers carried 1 mi. was 34,575,872,980. The corresponding return for 1912 was 1,536,762,172 less. The number of passengers carried 1 mi. per mile of road was 143,067, as against 140,393 for the preceding year.

The number of tons of freight carried was 2,058,035,487, while the corresponding figure for the previous year was 1,818,795,630, the increase being 239,239,857. The ton mileage was 301,398,752,108, an increase over 1912 of 37,618,843,854. The number of tons carried 1 mi. per mile of road was 1,245,158, as against 1,110,811 for the preceding year. The average number of tons of freight per train-mile was 445.45. The corresponding figure for the preceding year was 410.26.

The average receipts per passenger per mile, as computed for the year ended June 30, 1913, were 2.008 cents; the average receipts per ton per mile, 0.729 cent. The passenger-service train revenue per train-mile was $1.35.555; the freight revenue per train-mile was $3.24.347. The average operating revenues per train-mile were $2.45.387. The average operating expenses per train-mile were $1.70.374. The ratio of operating expenses to operating revenues was 69.44 per cent.

Revenues and Expenses.—The operating revenues of the railways represented (average mileage operated 242,657.12 miles) were $3,125,135,798; their operating expenses were $2,169,968,924. The corresponding returns for 1912 (average mileage operated 238,220.11 miles) were: Operating revenues, $2,826,958,366; operating expenses, $1,959,094,658. The following figures present a statement of the operating revenues for 1913 in detail:

OPERATING REVENUES

Freight revenue	$2,198,930,565
Passenger revenue	695,987,817
Excess baggage revenue	7,607,802
Parlor and chair car revenue	715,566
Mail revenue	50,789,212
Express revenue	79,717,266
Milk revenue (on passenger trains)	9,057,591
Other passenger revenue	6,110,252
Switching revenue	33,248,734
Special service-train revenue	1,980,362
Miscellaneous transportation revenue	6,861,901
Total revenue from operations other than transportation	31,628,843
Joint facilities—Dr.	1,054,003
Joint facilities—Cr.	3,553,890
Total operating revenues	$3,125,135,798

Operating expenses, as assigned to the five general classes, were:

OPERATING EXPENSES

Maintenance of way and structures	$421,232,395
Maintenance of equipment	511,561,363
Traffic expenses	62,850,113
Transportation expenses	1,096,252,745
General expenses	78,072,308
Total operating expenses	$2,169,968,924

With minor eliminations from the figures given above, operating revenues per mile of line operated (including line operated under trackage rights) averaged $12,873 and operating expenses $8,939 for the year.

Cost of Driving Piles with a Gasoline Hoist

Outfit Mounted on Scow and Used for Putting Down a Maximum of Seventy-five Piles Daily

A REVERSIBLE gasoline hoist with a 6½-hp engine and operating a 1650-lb. drophammer has been used for driving 1300 piles to support a stage for 7000 singers during the St. Louis pageant. These piles were driven from a scow about 6 ft. deep in the bottom of the Mississippi River at Forest Park. The niggerhead of the hoist was used to pull the piles in place and the drum was utilized for hoisting the ram. In addition, a pulley on the flywheel ran a centrifugal pump for keeping the scow dry.

In the accompanying diagram are shown the total number of piles to be driven on schedule and the actual number of piles driven, the estimated cost of driving 1300 piles and the actual cost of labor on piles driven. The largest number of piles driven in one day was seventy-five. In the estimate were included 17,105 lin. ft. of piling at a cost of 17 cents, giving a total cost of $2907.85. The average length per pile was estimated to be 13.15 ft.

Actually 1326 piles, aggregating 19,104 lin. ft. and averaging 14.4 ft., were driven. Of this number 25 piles were driven out of line, so that the useful number was 1301 piles, aggregating 18,735 lin. ft. Allowing 100 per cent depreciation on engine and scow, the cost of driving 18,735 lin. ft. was $2148.38, or 11.5 cents per foot. This depreciation, of course, is excessive, and if 20 per cent is allowed on engine and scow and 15 per cent for overhead charges the total cost of driving was $2000.92 or 10.7 cents per linear foot. The cost of the piles delivered was $1432.80, or 7.5 cents per linear foot, so that with a cost of driving of 10.7 cents the cost per linear foot of pile in place was 18.2 cents. The crew consisted of four men.

SCOW WITH GASOLINE PILE DRIVER

The itemized costs were as follows: Cost of piles delivered, $1432.80; total payroll, $1653.93; engine and hoisting outfit, $340; scow, $154.45.

The consulting engineer for the erection of this staging was Nelson Carnliff, of St. Louis, who also computed the costs. The machinery used was the Sultan gasoline reversible hoist manufactured by the Whitman Agricultural Company, of St. Louis.

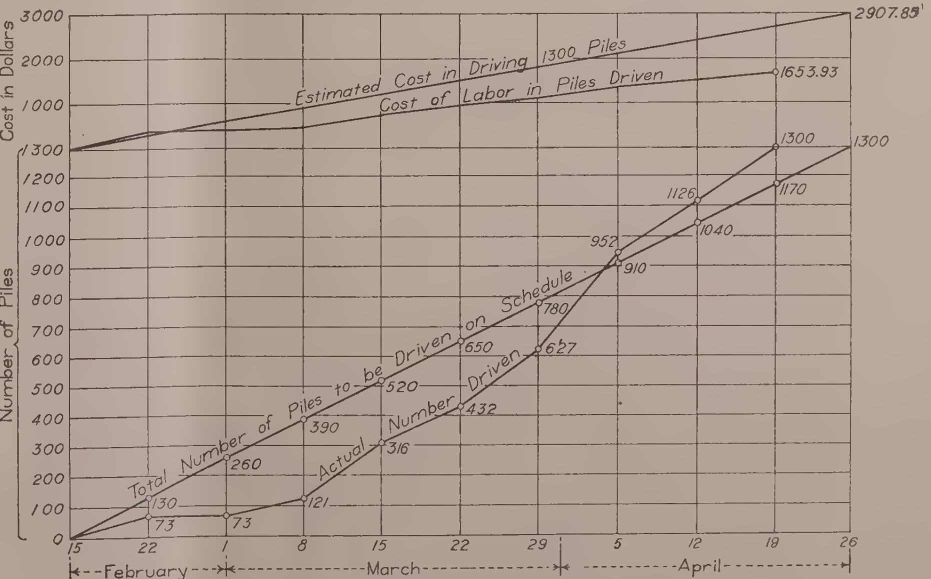


DIAGRAM OF SPEED AND COST OF DRIVING PILES WITH GASOLINE HOIST

WATER CONSUMPTION at Albany, N. Y., in 1913 was 220 gal. daily per capita, according to the annual report of the water bureau. There were 5756 meters. Slightly less water was wasted than in 1912 or 1911, when 242 gal. daily per capita were used.

American Society for Testing Materials

Additional Abstracts of Papers Read at the Annual Meeting in Atlantic City, June 30 to July 3

Volume Changes in Portland Cement and Concrete

By A. H. WHITE
Professor, University of Michigan

PORTLAND-CEMENT structures retain their colloidal property of expanding when wetted and contracting when dried, even after 20 years' exposure to the weather, and after baking to 600 deg. Fahr. A bar of neat cement may be made to expand or contract progressively, according to the choice of moisture conditions. It may be made by successive bakings to contract as much as 4.6 per cent. Cement bars containing sand do not contract readily on account of the interlocking of the aggregates. When such bars are dried they become more porous, and, therefore, on immersion more water can enter, so that new particles of cement clinker will be hydrated, with a still further resultant elongation of the bar. Bars of 1:3 sand mortar have by successive bakings and boilings been made to elongate 0.75 per cent. Bars cut from sidewalks taken up after 20 years' service have been made to elongate 0.175 per cent by successive immersions in water at room temperature. This elongation is as much as would be calculated for a rise in temperature of 300 deg. Fahr.

It must be considered a grave source of danger to concrete highways in wet locations. Dense concrete, even when wet, does not contain any liqud water and does not suffer any expansion on freezing. Such concrete when measured while frozen will be found to have contracted at the same rate as cast iron. Porous concrete containing liquid water expands on freezing and does not return to its original length on thawing because of the wedging of the aggregates. Repeated freezings will thus cause progressive disintegration.

Paint Protection for Portland-Cement Surfaces

By HENRY A. GARDNER
Assistant Director, The Institute of Industrial Research, Washington, D. C.

IN April, 1912 the writer instituted a series of tests to determine the durability of various types of paint upon Portland-cement surfaces exposed to the weather. The panels for the tests were prepared by constructing a long board wall to which was fastened expanded metal. A mixture of 1 part of Portland cement and 2 parts of clean Potomac River sand was made and applied to the expanded metal, forming a cement wall 3 in. in depth. The wall was divided into thirty-five sections or panels, each 30 in. wide and 40 in. high. Three coats of paint were applied to each panel by a practical journeyman painter. In order to make the test more severe, nearly all of the paints were applied in white. (Tinted paints are known to be much more durable than white paints.) A stripe of chrome green, 6 in. wide, was placed over the top of the third coat of paint, in order to determine whether the lime which might be present on the surface of the cement would have any effect upon the paint coating. Fading of the green to yellow would indicate such action.

The investigation had for its object the determination of what constitutes a suitable paint for the protection and decoration of exterior and interior stucco walls constructed of Portland cement and for decorating and rendering wearproof Portland-cement floors.

The tests showed that the amount of free lime in fairly dry cement surfaces does not adversely affect high-grade oil paints. The tests also showed that zinc sulphate may be used with excellent results as a primer to neutralize free lime in damp cement surfaces which are to be painted. The scaling of varnish paints, which developed in several tests, apparently showed that resinous paints are not well suited for cement surfaces.

Excellent results may be obtained by treating cement walls or floors with paints made with raw and heat-treated linseed and Chinese wood oil containing durable, wear-resisting pigments.

Strength of Lime Mortar

Limes Should Be Compared with a Standard Product and Only Relative Strength Given

By W. E. EMLEY and S. E. YOUNG
U. S. Bureau of Standards, Pittsburgh

THIS paper deals with an attempt to measure the compressive, tensile, shearing and transverse strengths of lime mortar and to ascertain how these values are affected by the numerous variables met with in practice, such as chemical composition of the lime, kind and amount of sand, size and shape of test specimen, conditions under which the test specimen is stored, the effect of added materials, etc.

The size and shape of a specimen of lime mortar are important factors influencing its strength, because the setting of the mortar depends upon the action of the carbon dioxide of the air. The size and shape are measures of the distance which this gas will have to penetrate in order to attack the interior of the mass. Another factor which influences the strength of lime mortar is the sand which it contains—both the amount of sand and the sizes of its grains. When a neat lime paste is permitted to set it shrinks so much as to cause pronounced cracking, and generally disintegration. The addition of a certain proportion of sand will overcome this tendency to crack and produce a mortar of measurable strength. Any further addition of sand will act merely as a diluent of the binding material and hence weaken the mortar.

The consistency of the mortar must also be considered. If the mortar is too dry it may be difficult to obtain a bond between the particles, and hence the specimen may be weak. If too wet it will be difficult to make the specimen homogeneous, and when the water evaporates high porosity may be expected. The amount of sand, as well as the amount of water, affects the consistency of the mortar. Hence the three component series, lime-sand-water, was investigated.

An important factor which must be considered is the condition of the atmosphere in which the specimens of lime mortar are stored. The setting of the mortar depends upon the chemical reaction between calcium hydroxide and carbon dioxide. Moreover, it is known that a certain amount of water is necessary to give the reaction measurable velocity, although too much water has the same effect as not enough water.

As a result of the studies it has been concluded that in order to use the strength of lime mortar as a basis of comparison every minute detail of manipulation used in measuring the strength must be specified. The extreme difficulty of maintaining uniform atmospheric conditions over long periods of time (which is absolutely essential if comparative results are to be obtained) practically precludes the specification of any definite figure for the desirable strength of a lime. It might be possible to eliminate the effect of the atmospheric conditions if some lime could be selected as a standard. Parallel series of specimens, one made of standard lime and the other of the lime to be tested, both subjected to the same conditions throughout, would give a means of comparing the unknown lime with the standard. Unfortunately, all commercial limes vary more or less in their properties, owing to the variations in the stone itself and in the temperature at which it is burned.

The strength of a lime mortar depends mainly on the atmospheric conditions and on the size and shape of the specimens. It is impossible to duplicate these practical conditions in the laboratory, and the present knowledge of the subject does not warrant the establishment of any definite relation between the strength as measured in the laboratory and that which may be expected in practice.

Rational Test for Metallic Protective Coatings

By JOHN A. CAPP
Engineer, Testing Laboratory, General Electric Company, Schenectady, N. Y.

THE problem of determining the resistance of protective coatings to weather corrosion is very similar to that of testing insulation for its weathering qualities. The conditions of exposure are the same, and hence there seemed to be no essential reason why the saturated atmosphere test would not only apply equally well to protective coatings as to insulation. Tests were begun several years ago to try out the method, and the only fault found with it was that it was somewhat slow.

As now used the test consists in exposing the articles in a convenient chamber, into which there is projected an atomized spray of water saturated with common salt in solution, care being taken to avoid placing the test specimens directly in the path of the jet. To insure constant saturation, an excess of salt is produced by a jet of compressed air lifting the water to the nozzle, whence it is projected as a cloud.

The salt-spray test has been used in the laboratory with which the author is connected for a number of years, and during the last four or five years it has also been used commercially as a check upon the process of sherardizing which is in use. When the coated material is iron in any of its several forms, red rust begins to develop as soon as the coat breaks down. This rust may appear in small pin points, which gradually extend, or it may appear over the entire surface of the article. When the coating is relatively thin and poor, rust may develop in from 2 or 3 hr. to 24 hr., or longer. A better coat will last two or three days, but a well-applied coat of requisite thickness will last at least a week. If no rusting is developed in two weeks' time it may safely be assumed that the life of the coating will be practically indefinite. These figures are based on experience with both sherardized and galvanized types of coating. Other types give results which lie in approximately the same range.

Use of Turned Sections in Tension Tests of Reinforcing Bars

Machining of Specimen Is Shown to Increase the Strength of the Material Up to 10 Per Cent

By E. P. WITHROW and L. C. NIEDNER

SINCE the standard specifications of the society for billet-steel concrete reinforcement bars permit the use of turned test specimens of deformed bars it is thought desirable to submit certain facts for consideration.

In a qualitative but not quantitative study of the effects of turning specimens of reinforcing bars it is found that there is an increase in the yield point and ultimate strength which may amount to 10 per cent. This is true of constant-section deformed bars and even of plain bars.

Another factor—namely, rate of application of load—cannot be neglected, because working conditions in most laboratories will introduce it secondarily when reduced-area specimens are tested. This factor can cause a variation of indicated yield point and ultimate strength of 1 or 2 per cent. As this amount is of the same magnitude as the percentage of steel in some types of deformations it should not be neglected. From the data given by Campbell an increase of 100 per cent in the speed of the pulling-head could cause an increase of 1 or 2 per cent in yield point. Therefore the average rate of application of stress was kept constant up to the yield point in the tests on which these conclusions are based.

BEHAVIOR OF YIELD POINT

On examination of the data the behavior of the yield point of specimens embedded in concrete was found to agree closely with that of the same steel held in wedge grips.

The appearance of a 1-in. plain square bar which was being examined microscopically suggested non-uniformity of metal as one of the causes contributing to the raising of the elastic limit by machining specimens. There has been no attempt to determine the magnitude of the effects on the physical properties caused by segregation of various elements. It has been shown, however, that large and variable differences may exist and that these differences, probably due to non-uniformity of steel, may, even in a constant-section bar, be greater than the reduction of section due to the removal of inactive deformations.

The increase in strength of turned specimens is not due to protection from the grips by the large ends of turned specimens for the following reasons: (1) If the specimens are turned to the same diameter for their full length the increase still exists; (2) the yield points of bars embedded in concrete at each end (the stress being applied through the concrete) agrees, approximately at least, with that of the same bar held in wedge grips. The indications are that the increase is caused by the chemical and structural non-uniformity of the bars. Since the removal of the exterior steel tends to give a higher reported value for the bars than they would develop in service, it is not good practice to allow the machining of test specimens of reinforcing bars.

Concrete Tank for Smudge-Pot Distillate in California

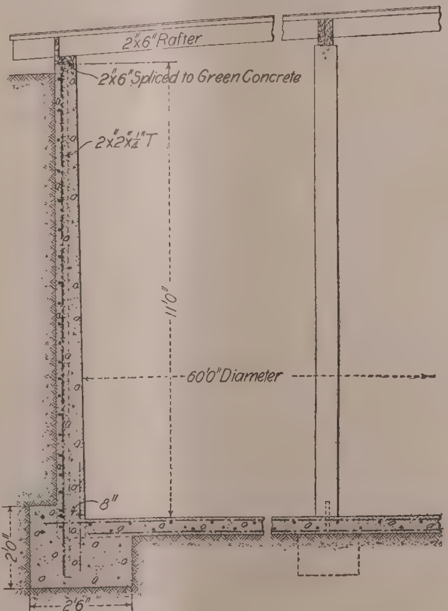
Reservoir Oil-Proofed by Adding 10 Per Cent. of Hydrated Lime to Concrete

ABSOLUTE assurance of having material on hand for smudging in the citrus belt of Southern California, when a falling temperature requires this measure, has led to the storage on the ground of "slop distillate," a local name for the low-grade oil used in the smudge pots. Many supplies for the individual grower are handled by the association which packs and sells his output. At Narod the West Ontario Citrus Association has recently constructed a reinforced-concrete distillate reservoir of 235,000-gal. capacity, sufficient to carry the orchards over the low-temperature period. It is 60 ft. in diameter and 11 ft. deep.

Many attempts have been made with varying degrees of success in building reservoirs of concrete which would be impervious to oil, as even the heavy grades of crude oil will permeate walls which are watertight, but the lighter grades of oil and distillate are harder to confine. The engineers report that the Narod tank has been filled with distillate for more than six months without any evidence of seepage.

A section of the wall is shown herewith. Eight 6 x 6-in. redwood posts, 15 ft. from the center post, support the composition roof laid on top of ⅞-in. sheathing.

The floor is reinforced top and bottom with ¼-in. rods spaced on 12-in. centers both ways. The concrete block under the wall extends 8 in. beyond the outer edge of the wall and has a rim against which the 8-in. non-battered wall abuts. Horizontal wall joints at the end of a day's run consisted of a bond of No. 27 galvanized iron sheet 6 in. wide, embedded half its width in each day's work.



SECTION THROUGH WALL OF NAROD TANK

Oil-proofing was accomplished by adding 10 per cent of hydrated lime to the concrete during the mixing operation. In addition the entire inner surface was given a plaster coat of "gunite" from ⅜ to ½ in. thick, applied with a cement gun.

Mayberry & Parker, architectural engineers, of Los Angeles, designed the tank, which was built by L. Fleming, of Pomona, Cal.

Street Repairs in Milwaukee

Costs Per Square Yard of Macadam Reduced from 31 Cents in 1912 to 11.2 Cents in 1913

STREET repairs in Milwaukee cost per square yard 11.2 cents in 1913 for macadam, being reduced from 31 cents for the previous year, when the work was done by ward gangs. In 1913 special gangs were organized. Stone and block paving repairs cost 83 cents per square yard. Asphalt repairs in 1908-1909 cost $1.50; in 1910-1911, about $1.57; in 1912, $0.845, and in 1913, $0.681. Interesting reading follows in the annual report of Department of Public Works on the inability to obtain more than 275 sq. yd. per day of asphalt from a portable plant of 1000-sq. yd. rated capacity, which refused to move from a firm stand taken under a viaduct.

Omission of the binder course from asphalt pavements in an effort to cheapen the work in Milwaukee was not a success. Observation in 1912 indicated that pavements constructed on smooth pavements did not push as much as a pavement on a rough concrete and that no bond existed between the binder course and concrete. Following this observation the 2-in. wearing surface was laid directly on the surface. To quote the report: "The wearing surface, however, apparently in resentment, immediately began to wriggle and squirm, until the surface assumed the billowy appearance of the ever-rolling and deep-blue sea." The conclusion was that a 2-in. depth does not possess sufficient stiffness to resist pushing, while 3 in. does except under very heavy traffic. About 25 per cent of the season's repairs consisted in repairs on this binderless pavement, none of which was down to exceed two years.

SIDEWALK REPLACING

Sidewalk replacing, placed in the hands of three city gangs of cement workers, was done at a cost of 11.8 cents per square foot. Only two gangs were supplied with gasoline mixers, but another mixer to be added this season will probably reduce the cost to 10.5 cents per square foot.

About 5000 cu. yd. of crushed stone were prepared and delivered in the street at 86.5 cents per cubic yard. During 1914 the crusher will not be operated, but large storage piles will be stocked in four or five convenient places, inasmuch as the average amount crushed in any one location did not exceed 500 cu. yd. The cost of moving indicated it would be economy to discontinue crushing until the large stock piles of uncrushed material had been accumulated.

Frank W. Blodgett is superintendent of steel construction, reporting to Fred G. Simmons, commissioner of public works.

Removal of Carbonic Acid, Iron and Manganese from Lowell Well-Water Supply

Frank A. Barbour Recommends Plant for Treatment by Limited Aeration, Coke Prefilters and Final Sand Filters, to Cost $180,000

REMOVAL of carbonic acid from one of the two well-water supplies of Lowell, Mass., and of iron and manganese from the other is to be accomplished—if the recommendations of Frank A. Barbour, consulting engineer, of Boston, are followed—by a plant involving aërators, six coke prefilters, 10 ft. deep, a 500,000-gal. settling basin, six sand filters and a 1,000,000-gal. filtered-water reservoir. The proposed works are to have a daily output of from 7,500,000 to 8,500,000 gal. and will cost $180,000. The average operating expense, including interest, depreciation, maintenance of the plant, and the cost of extra pumping, is estimated to be $7.65 per 1,000,000 gal. Mr. Barbour's report was submitted to Commissioner James H. Carmichael June 4, of which the following is an abstract:

WELL-WATER TROUBLES

Lowell has two sources of supply—one on the north bank of the Merrimac River and the other in the valley of River Meadow Brook. Both are ground-water supplies, drawn through driven wells. That on the Merrimac, known as the Boulevard system, furnishes the greater part of the water used; the other, which includes two gangs of driven wells—the Cook and the hydraulic —is only brought into service when inadequacy of the Boulevard supply makes this necessary, for the reason that in 1899 the State Board of Health condemned the Cook wells because of the action of this water on lead pipe, and the consequent danger of poisoning. With continued use, and to a constantly increasing degree, the Boulevard supply has grown unsatisfactory owing to the amount of iron and manganese in this water, which renders it turbid and altogether disagreeable. This, then, in short, is the present condition of the water supplies of the city—one source condemned because of the action of the water on lead pipe, and the consequent danger of poisoning and the other so highly charged with metallic contents as to make it unfit for use.

The present ground-water supplies, as is proved by the low typhoid-death rate of the city, have always been satisfactory in point of freedom from bacterial contamination, and the present difficulties are simply chargeable to the high carbonic-acid content of the Cook well water, which attacks the lead service pipes, and to the iron and manganese in the Boulevard supply, which render this water turbid and disagreeable. Naturally therefore, the larger part of the investigation has been devoted to experimental work to determine the feasibility of so treating the present ground-water supplies as to make them satisfactory.

The experimental work proved that the Cook well water could be made safe and suitable for use by aëration at reasonable cost and rendered fit to serve as an auxiliary to the Boulevard supply, if required.

To determine whether the iron and manganese could be satisfactorily and economically removed from the Boulevard water an experimental plant has been operated during the past eight months. The experiments were intended to determine, on a practical scale, the basis of the design of an actual plant, and to make certain, before undertaking the construction of such a plant, that it would work successfully. The experimental plant consisted of a prefilter of gravel or coke and a final sand filter. The experimental work proved that the iron and manganese can be successfully removed from the Boulevard water at reasonable cost. The necessary method was found to require a coke prefilter not less than 8 ft. deep, operated as a contact bed at a rate of between 60,000,000 and 70,000,000 gal. per acre daily, and a sand filter operated at a rate of 10,000,000 gal. daily. By this process the iron and manganese were each reduced to less than 0.12 part per million in the final effluent. It was also proved that a lesser depth of the prefilter, or too much aëration, or the operation of the prefilter as a trickler, was not adapted to the treatment of the local water.

THREE COURSES OPEN

Three courses were open to Lowell in seeking an improved supply: First, treatment of the present supplies so as to render them safe and satisfactory; second, filtration of the Merrimac River; third, connection with the metropolitan system.

For filtering the Merrimac River water it is estimated that the first cost of a plant at the Boulevard would approximate $250,000, and that the average cost of filtering this water, including interest, depreciation and operating charges, would be $10.27 per 1,000,000 gal., or $2.62 more than the cost of treating the ground water. When, however, the expense of maintaining the well system—estimated at $4.40 per 1,000,000 gal.—is added to the cost of the ground-water supply it follows that the total is greater than that of filtering the Merrimac River by $1.78 per 1,000,000 gal.

Because of the greater safety of the ground water, the lesser color, the lower temperature in summer, and the popular prejudice against the use of a polluted river water, however well filtered, it is concluded that this small difference in the cost is not of moment, and that, so long as Lowell can continue to obtain an adequate supply from the ground at such reasonable cost as has been shown to be possible, this should be done.

A study has also been made of the possibility of connecting with the metropolitan supply at the Wachusett dam, and it has been found that, on the basis of the cheapest reasonable development, this connection would involve a first expenditure of not less than $1,750,000 and a final cost of not less than $60 per 1,000,000 gal. of water taken, or practically double the cost of maintaining and purifying the present ground-water supply, or of filtering the Merrimac River water.

It is therefore finally concluded that the city should further develop the Boulevard system of ground water, and should immediately undertake the construction of works for the removal of the iron and manganese, which alone prevent this supply from being entirely satisfactory.

Experiments showed that the decarbonation of the Cook well water by treatment with lime was undesirable and uneconomical because of the resulting increase in hardness. Results in removing the carbonic acid by aëration with spraying nozzles proved satisfactory.

The tests indicated that by a pressure of 5 lb. on the nozzle the carbonic acid could be reduced from 45 parts per million in the raw water to about 3.3 parts per million in the aërated water, and that with this reduction the effect of the water on lead pipes was so far lessened as to render the supply safe. In stating this conclusion the fact that in the 15-hr. tests the lead taken up somewhat exceeded 0.5 part per million is not overlooked, because it is believed that the retention of the water for 15 hr. in the pipe is too severe a test, and that the 4-hr. period of contact more nearly represents the conditions of actual use.

The removal of carbonic acid by aëration has a great advantage over the lime treatment in that it does not increase the hardness, and further, as indicated by the tests, aëration is more effective than the lime treatment in reducing the corrosive action when an equal amount of carbonic acid is left in the treated water. This result is not what might be expected, because by aëration the dissolved oxygen, which has generally been considered a contributory factor in corrosion, is increased; but the experiments clearly indicate that, with an equal content of carbonic acid, the aërated water takes up less lead than the lime-treated water.

DEFERRIZATION AND DEMANGANIZATION

The plant for the deferrization and demanganization experiments on the Boulevard well water consisted of a coke filter, 20 in. square, in which the depth of the material could be varied, and a sand filter 30 in. square.

In the first tests the depth of coke in the prefilter had been 3 ft.; it was later increased to 4.5 ft. and finally to 7 ft. In experiment 16, from Jan. 16 to 25, 1914, the water was aërated, the prefilter operated as a trickler at the rate of 22,500,000 gal. per acre daily, and the sand filter at a rate of 10,000,000 gal. per acre daily. The immediate effect of increasing the depth of the prefilter was the lengthening of the run of the sand filter before it became necessary to scrape, and while, in previous tests, the amount filtered between scrapings averaged about 30,000,000 gal. per acre, in this experiment 93,500,000 gal. per acre were passed through the sand before cleaning.

Table 1 shows the average results of this run.

TABLE 1—RESULTS OF EXPERIMENT 16

Determination	Raw	Coke prefilter	Sand filter effluent
Iron (p. p. m.)......	1.920	0.660	0.600 — 0.200
Manganese (p. p. m.)	2.100	1.800	1.050
Carbonic acid (p. p. m.)	24.5	4.5	8.0
Dis. oxygen (per cent sat.)	14.1	97.1	84.9

The dissolved oxygen content of the raw water averaged 14 per cent of saturation; the prefilter effluent was practically saturated, and the final effluent contained 85 per cent of dissolved oxygen. The final effluent had a blue tinge throughout the test, and the reduction in iron and manganese was not satisfactory. Upon cleaning the sand filter the surface deposit was found to be black in color and capable of being rolled up in a thin sheet. Below this tough surface layer the sand was discolored and tightly caked together by a black manganese precipitate for a depth of 3 in.

The conditions of experiment 17, from Jan. 27 to Feb. 14, 1914, were the same as those of experiment 16, except that the depth of coke prefilter was 7 ft. throughout the run. Again, the increased amount filtered before scraping became necessary, due to the greater depth of coke, is noticeable—177,000,000 gal. per acre being filtered in a period of 428 hr.

Table 2 shows the average results of experiment 17.

TABLE 2—RESULTS OF EXPERIMENT 17

Determination	Raw	Coke prefilter	Sand filter effluent
Iron (p. p. m.)......	2.025	0.495	0.400 — 0.215
Manganese (p. p. m.)	3.280	2.630	1.600 — 0.100
Carbonic acid (p. p. m.)	24.3	4.8	8.4
Dis. oxygen (per cent sat.)	13.6	101.7	86.0

During the greater part of this test the final effluent had the characteristic bluish tinge, due to colloidal iron, and the amount of manganese remaining in the treated water was too great to be considered satisfactory. When the sand filter was cleaned, the surface deposit was found to be black, and the pronounced discoloration by the manganese precipitate extended to a depth of 2 in. and in a minor degree entirely through the bed and into the underdrains.

Before beginning experiment 18, from Feb. 16 to April 25, 1914, the sand filter was refilled with clean, washed sand, 24 in. deep, of the same quality as used in the earlier runs. The aërator was cut out, but as the prefilter was only backflooded for a depth of 2½ ft., from Feb. 16 to 27—the upper 4½ ft. operating as a trickler—aëration during this period was practically complete, the prefilter effluent being 95 per cent saturated. It is noticeable that during this part of the run the final effluent had the bluish opalescence. After Feb. 27 the prefilter was backflooded 6½ ft., thus increasing the time of contact with the coke and reducing the aëration, and with this change the character of the final filtrate became entirely satisfactory.

Table 3 shows the average results of experiment 18.

From Feb. 16 to March 24 the coke prefilter was operated at a rate of 22,500,000 gal. per acre daily—the depth of backflooding being, as already stated, 2½ ft. from the beginning of the run to Feb. 26, and 6½ ft. thereafter. From March 25 to April 16 the prefilter was operated at a rate of 45,000,000 gal. per acre daily, and from April 17 to the end of the run, at a rate of 67,500,000 gal. per acre daily. The increasing of the prefilter rate from 22,500,000 to 67,500,000 gal. per acre daily had no apparent effect on the quality of the effluents from the prefilter or final sand filter.

TABLE 3—RESULTS OF EXPERIMENT 18

Determination	Raw	Coke prefilter	Sand filter effluent
February 16-26			
Iron (p. p. m.).........	1.600	0.600	0.315
Manganese (p. p. m.)...	3.230	2.200	1.570
Carbonic acid (p. p. m.).	24.5	7.4	6.3
Dis. oxygen (per cent sat.)	16.6	93.0	86.6
February 27-*March* 20			
Iron (p. p. m.).........	1.400	0.420	0.165
Manganese (p. p. m.)...	3.090	1.860	0.590
Carbonic acid (p. p. m.).	20.5	12.3	14.6
Dis. oxygen (per cent sat.)	13.2	50.4	41.5
March 21-24			
Iron (p. p. m.).........	1.400	0.360	0.250
Manganese (p. p. m.)...	2.480	1.080	0.010
Carbonic acid (p. p. m.).	22.8	15.8	15.7
Dis. oxygen (per cent sat.)	11.8	37.3	31.7
March 25-*April* 16			
Iron (p. p. m.).........	1.250	0.280	0.140
Manganese (p. p. m.)...	2.100	1.050	0.010
Carbonic acid (p. p. m.).	23.3	14.0	14.5
Dis. oxygen (per cent sat.)	11.8	34.0	25.0
April 17-25			
Iron (p. p. m.).........	1.050	0.300	0.090
Manganese (p. p. m.)...	2.100	1.150	0.010
Carbonic acid (p. p. m.).	23.2	9.6	13.8
Dis. oxygen (per cent sat.)	13.5	45.2	35.4

On March 20, when 279,000,000 gal. per acre had been filtered and the loss of head had reached the maximum, the sand filter was raked, but no material was removed. Afterward the run was continued until April 25, when 311,000,000 gal. per acre additional had been filtered, or a total of 590,000,000 gal. per acre, before it became necessary to scrape.

That the preliminary treatment in this experiment was capable of so precipitating the iron and manganese as to cause these metals to be caught at the surface of the sand is evidenced by the fact that almost the entire loss of head occurs in passing through the surface layer.

AMOUNT OF DISSOLVED OXYGEN

It is to be noted that, after Feb. 27, when the results of this experiment may be said to have been entirely successful, the dissolved oxygen in the prefilter effluent did not exceed 50 per cent of saturation, and this condition—namely, that limited aëration, or the retention of a considerable part of the carbonic acid, is necessary to the successful treatment of the Boulevard water—may be considered to be clearly indicated by all the experiments.

The prefilter removed from 40 to 60 per cent of the manganese and from 55 to 80 per cent of the iron. The combined plant during the last month of the experiment removed 90 per cent of the iron and 99.5 per cent of the manganese, the actual average quantities of these metals remaining in the final filtrate being 0.11 part per million of iron and 0.01 part per million of manganese. No better results could be desired, and when it is considered that during the experiment 590,000,000 gal. per acre were filtered before scraping became necessary, it is evident that the process was not only efficient, but economical in point of maintenance.

In considering the amounts of iron and manganese in the final effluent, it may be of interest to note that observations of samples kept for several days indicated that with less than 0.30 part of iron per million no precipitation, either in the cold or when the water was heated, occurred, but that, with 0.30 part per million of iron, a precipitate sometimes formed, and at other times did not develop, depending, apparently, on the character of the treatment of the water preliminary to filtration through the sand, the more stable condition following a low oxygen content in the prefilter effluent. When the bluish tinge was present in the final effluent, precipitation occurred in almost all cases after several days.

SUMMARY AND CONCLUSIONS

The experiments indicate that the iron and manganese in the Boulevard water cannot be removed by aëration and direct application to sand filters, nor by aëration, sedimentation and sand filters; that excessive aëration and the operation of the prefilter as a trickler are not adapted to the treatment of the local water; that the iron and manganese can be successfully and economically removed by limited aëration, passage through a coke prefilter not less than 8 ft. in depth, operated as a contact bed at a rate of 67,500,000 gal. per acre daily, and subsequent filtration through sand at a rate of 10,000,000 gal. per acre daily.

On these conclusions plans of a purification plant have been prepared, based on the use of coke prefilters 10 ft. deep, an intermediate sedimentation basin of 1 hour's capacity, and sand filters 3 ft. deep. The depths of both filters are greater than in the experimental apparatus, and the addition of the intermediate sedimentation basin will serve to reduce the work to be done by the sand filter. It is believed the experiments were sufficiently prolonged, and on such a scale as to permit safe conclusions to be drawn, and that there can be no doubt that the plant designed will successfully remove the iron and manganese and render the Boulevard supply entirely satisfactory.

Difficult Bucket-Well Construction

Concrete Lined Pit Excavated and Sealed under Water in Sheet-Pile Cofferdam in Quicksand

THE terminal coaling station of the Chicago, Milwaukee & St. Paul Railway Company at Council Bluffs, Iowa, is of the elevator type with storage bins filled by two 1-yd. buckets operating in wells about 18 ft. deep. On account of the unstable character of the wet soil the wells were sunk by means of an open pit 21 ft. deep, made by submerged excavation with a dredge bucket operated in a sheet pile cofferdam driven below the bottom of the pit and sealed by concrete deposited under water. This method maintained balanced pressure on the quicksand most of the time, dispensed with foundation piles, and allowed the walls to be built in the dry without difficulty.

The soil consists of alluvial deposit to a depth of about 16 ft., below which there is a stratum of quicksand 90 ft. deep. The ground-water level varies with the height of the nearby Missouri River, rising sometimes to within 4 ft. of the surface, but at the time that this work was executed it was about 9 ft. below the surface. When the quicksand is undisturbed by the passing of water through it, it is hard and compact. Under the influence of water it flows readily when unconfined. It was at first thought desirable to drive piles for the foundation of the concrete bottom and well lining, but it was eventually decided to omit them. The bottom rests on the surface of the undisturbed quicksand, at a depth so great that it is believed to be beyond disturbance by future excavations in the vicinity.

EXCAVATION THROUGH QUICKSAND

The top soil was excavated and excavation through quicksand was commenced by hand in a cofferdam having 2-in. tongue and groove sheeting and 6 x 8-in. walings and cross timbers. In this cofferdam was driven a 16 x 37-ft. second cofferdam, containing both bucket wells. The 6 x 10-in. Wakefield sheet piles, 16 ft. long, were driven by a 2200-lb. drop hammer suspended in leads carried by the 50-ft. boom of a stiff-leg derrick. The cofferdam was provided with three tiers of rangers braced by horizontal transverse struts and having four kneebraces to the center points of the sides of each of the two panels thus formed. All of the rangers and braces were of 8 x 16-in. timbers adjusted by wedges at the ends of the kneebraces.

The quicksand excavation was completed with a ½-yd. clam-shell bucket operated by the derrick boom. To avoid as much as possible the inflow of quicksand, the dredging was done under water, and the pit was pumped out only when it was necessary to set the braces, or to clean out the corners where the dredge bucket could not reach. For these purposes the pit was drained by a 6-in. direct-connected centrifugal pump, the operation of which caused large quantities of quicksand to flow into the excavation. The sheeting was driven to a depth of 25 ft. below the surface of the ground before excavation with the clam-shell bucket was commenced.

After the excavation was completed to a depth of 21 ft. below the surface of the ground, the pit was allowed to fill with water up to ground-water level and 1:2½:5 concrete was deposited under water with a ½-yd. conical bottom-dump bucket, originally designed for dumping concrete into narrow wall-forms. A mass of concrete 3 ft. thick was thus placed and after it had set the pit was pumped out and found to be so well sealed that all quicksand was excluded and the leakage water was handled by a No. 5 injector. The remainder of the concrete for the lining and floor was mixed 1:2:4 and built in forms in the usual manner.

The work was executed by the railroad company's forces under the direction of A. P. Munsen, assistant engineer.

Making Dynamite Work Efficiently

APROPOS of the article in the Engineering Record of July 4, page 8, under the above title, a correspondent recites an experience at Valhalla, N. Y. Wagon drills were used on the work, and when first employed the holes were spaced 6 ft. on centers. Finally, at the suggestion of the manufacturer of the drills, the center distance was increased until it was found that holes on 30-ft. centers, with holes well sprung, gave much better results. This meant that under the latter arranagement one hole was doing better work than was formerly done by nine. The cost of springing was almost negligible when taking into consideration the saving in labor, time, power, etc.

LIVE-LOAD STRESSES of the steel in the Holton Street viaduct, Milwaukee, were measured last spring by the superintendent of bridges, J. C. Pinney, Jr., by a recording extensometer. They did not exceed 2100 lb. per square inch and agreed closely with computed stresses.

Concrete Road Broken by Frost Upheaval

Well-Laid One-Course Pavement at North Andover, Mass., on Poorly Drained Foundation Was Fractured During First Winter

UNLESS he knew the conditions under which the road was laid, the average observer viewing some badly broken concrete sections in North Andover, Mass., during the past spring would be likely to think the experience a blow to the cause of the concrete highway. Many sections were badly cracked and for about 2000 ft. the slabs were raised, in some instances as much as 6 in. These results, however, were not entirely unexpected, as the road—2 mi. of it—was built as an experiment. The soil and water conditions over most of the length were extremely adverse. For instance, at the point shown in two of the photographs the ground-water level was about even with the surface of the concrete. Had the State highway department, which built the road, provided a foundation and a drainage system competent to minimize the possibility of frost action, it would perhaps have doubled the cost. It was decided to lay the pavement under these unfavorable conditions, using different brands of cement, varying the cross-section and the type of expansion joint, using two mixtures of concrete and reinforcing some sections with wire mesh.

Laid in the fall of 1913, special care seems to have been taken in the building of the road, which is 19½ ft. wide. Of the three cross-sections, the first, 6 in. thick at the edges and 8½ in. at the center, and the second, 5 in. at the edges and 7½ in. at the center, are flat bottomed. The third section is 6 in. thick throughout, with the bottom crowned parallel to the top. The slabs are about 30 ft. long, separated with expansion joints, half of them Baker plates with tar felt between, and the other half simply tar felt. Where reinforcement was used it was placed 2 in. above the bottom of the slab. The largest broken stone used was 2½ in.

About 1000 ft. east of Wilson's Corner and extending easterly about 2000 ft., a stretch where drainage conditions were particularly bad, 6 in. of sandy gravel were placed under the slabs. Side drains were also laid on the northeast side of the road about 4000 ft. east of that end.

Considered without respect to the inferior foundation, construction of the road appears to have followed the best practice. Mixing, placing, finishing and brooming were carefully watched, and the finished surface was covered with earth and kept wet eight days. The crown at two of the cross-sections was ¼ in. to the foot and for the third was ½ in. to the foot. Two of the accompanying views show the road under construction. Although the officials of the highway department do not feel that the experience of the road has yet been sufficient to warrant a report on the performances of the different sections, it may be noted that two or three of the reinforced sections cracked and heaved badly.

The displaced slabs, which returned to their original positions as soon as the frost was out of the ground, were raised in many places as much as half a foot, probably not less than 0.1 ft. anywhere, and often from 0.1 to 0.2 ft. on one side and 0.4 to 0.5 in the center. After the slabs subsided the cracks were repaired with hot tar and hot sand, the repairs costing about $25 for the half mile involved. Had it been desired to replace the worst sections of the road only about 1000 ft. would have had to be removed, but this will, of course, not be done, as the commission wishes to observe the results of future frost action.

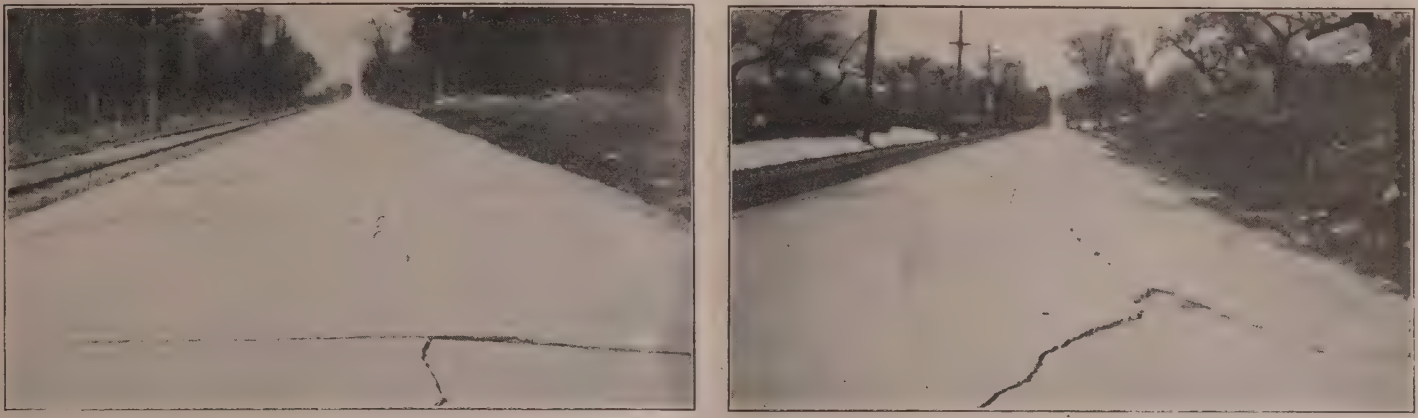
Costs

As this is an experimental road, any costs that may be given will not be accurately representative of ordinary work, but the figures for the various elements of the construction may be discounted for this condition. The contractor, the David J. Sheehan Company, of Lynn, Mass., contracted for all the labor, as well as paying for sand, stone, and tar paper, the State supplying cement, armor plates and reinforcement. The actual cost to the contractor for the 6-in. section, using a 1:2:4 mix, was 73.4 cents per square yard—28.9 cents for labor, 44.4 cents for stone, and 0.1 cent for tar paper. The contract price was 79 cents and the area involved was 15,201 sq. yd. The unit cost to the State of this section, not figuring overhead charges, was $1.214, the additional unit prices being: cement, 43.1 cents, and armor plate, 4.9 cents. On a cubic-yard basis this cost figures $7.15.

Concrete laid in accordance with the other cross-sections was all of 1:1½:3 mix. The cost to the contractor was 78.3 cents and

TROWELING SURFACE PREVIOUS TO BROOMING

BEVELING EDGE OF FINISHED SURFACE



VIEWS OF TWO SECTIONS OF CONCRETE ROAD IN MASSACHUSETTS, CRACKED BY FROST ACTION IN SATURATED FOUNDATION

the contract price was $1 per square yard. The actual cost to the State, not including overhead, was $1.587, distributed as follows: Labor, 30 cents; sand and stone, 48.2 cents; cement, 63.6 cents; armor plate, 4.6 cents; tar paper, 0.1 cent, and wire-mesh reinforcement, 12.2 cents. The area of this section was 8082 sq. yd. On the cubic-yard basis the cost was $7.97.

Excavation costs, distributed over the total area, 22,779 sq. yd., are given in the accompanying table.

TABLE OF EXCAVATION COSTS.

Item	Quantity	Rate	Amount
Foremen	315 hr.	33 1/3 cts.	$105.00
Labor	4352 hr.	22 2/9 cts.	967.11
Teams	990 hr.	55 5/9 cts.	555.00
Steam Roller	25 days	$12.	300.00
Total			$1927.11

Cost per square yard, 8.46 cents.

That the officials of the State highway commission believe that the experience of this road has been educational and not a cause of embarrassment was shown on May 14, when, during a tour of inspection of the Massachusetts Highway Association, special emphasis was placed on the facts concerning this pavement. Its performance through next winter and the succeeding spring will be watched, and reports on the experiment will probably be made at that time.

N. Y. State's Policy on Problems of Water and Sewage

Special Commission Answers Specific Questions Relating to Protection of Watersheds

IN view of a number of pressing problems as to the location of tuberculosis sanatoria and other public institutions upon watersheds used as gathering grounds for public water supplies and involving the general policy of the New York State Department of Health toward sewage-disposal plants and water supplies throughout the State, Commissioner Hermann M. Biggs appointed a special commission to answer certain specific questions and to make such other general recommendations as it might deem proper. The commission consisted of Dr. T. M. Prudden, vice-president of the Rockefeller Institute for Medical Research, chairman; Prof. H. N. Ogden, professor of sanitary engineering in Cornell University; Dr. W. H. Park, director, research laboratories, New York City Health Department; Prof. G. C. Whipple, professor of sanitary engineering in Harvard University and the Massachusetts Institute of Technology; and Prof. C.-E. A. Winslow, professor of biology in the College of the City of New York. The commission met on May 9 and unanimously reported the following replies and recommendations:

ANSWERS TO SPECIFIC QUESTIONS

Q. Can the sewage of any institution, whatever its character—hospital, home, asylum, reformatory or prison, for example—be safely disposed of, so that it shall not be a source of risk either to its inmates or to the community, near or remote, provided that there is sufficient space and funds are available for the installation of an adequate system, which is properly administered and subject to the necessary supervision and inspection, and provided further that provision be made for the repair or renewal of the system when these become necessary? A. It can.

Q. Should the State Department of Health make specific requirements for the construction and administration of such sewage-disposal plants as it may permit to be installed in cases of institutions in which special safeguards of sewage are required? A. Yes.

Q. Should all sewage-disposal plants in the State of New York, whether of private houses, institutions, or municipalities, which discharge their effluents into drainage areas which are feeders for public or private water supplies, be subject to the official supervision of the State Department of Health, by periodic examinations of the effluent, and by inspections of plant, and operation or otherwise, and to official control as to efficiency of the plant? A. Yes.

Q. Should the State Department of Health have more effective power to compel the installation of an efficient sewage-disposal plant for the purification of sewage, the discharge of which appears otherwise to be a nuisance or a menace to health? A. Yes.

Q. Is it, in the opinion of the commission, safe for any institution, municipality or community to use a water supply derived from surface drainage of an inhabited region, which is liable to contamination from house or institutional or community waste, or from occasional surface contamination from roadways and railroads, or in other ways, even though limited areas along the main drainage channels and reservoirs are sequestered and protected, unless such water supply has been safeguarded by adequate purification, sufficiently effective to remove or to destroy pathogenic bacteria? A. No.

Q. Assuming that the commission shall decide that any or all of the designated sites can be safely used for the specified purposes, should adequate routine laboratory examination and local inspection be required of the authorities of the institution in question, in order to assure the continued efficiency of the proposed safeguards when these are installed, reports of such examination and inspection to be regularly made to the State Department of Health? A. Yes.

Q. Should examinations and inspections also be made regularly by the State Department of Health at such intervals as it may deem necessary? A. Yes.

RECOMMENDATIONS IN REGARD TO SPECIFIC PROBLEMS

In regard to a proposed site for a tuberculosis hospital in North Castle, Westchester County, the commission recommends that the commissioner approve the application, provided that proper provision is made for the safe disposal in the Mianus River watershed of the sewage from the institution; the treatment of such sewage to be by intermittent filtration, at a suitable rate satisfactory to the health commissioner, and disinfection coupled with such preliminary processes as may be necessary; the design, construction and operation of such sewage-disposal system to be satisfactory to the State Department of Health.

In regard to proposed changes in the sewage-disposal plant of the Bedford State Reformatory for Women the commission recommends that the commissioner permit the disposal of the sewage of this institution within the catchment area of Broad Brook, provided that proper provision is made for the treatment of such sewage, by intermittent filtration, at a suitable rate satisfactory to the health commissioner, and disinfection, coupled with such preliminary processes as may be necessary; the design, construction and operation of such sewage disposal system to be satisfactory to the State Department of Health.

In regard to the two preceding special cases, the commission recommends that in the foregoing cases the commissioner should require such weekly reports of operating data as he may prescribe, on forms provided by the department.

GENERAL RECOMMENDATIONS

The commission makes the following general recommendations:

The State Department of Health should have power to require such improvements as it may deem necessary for the protection of the public health in the construction and operation of any system of public water supply.

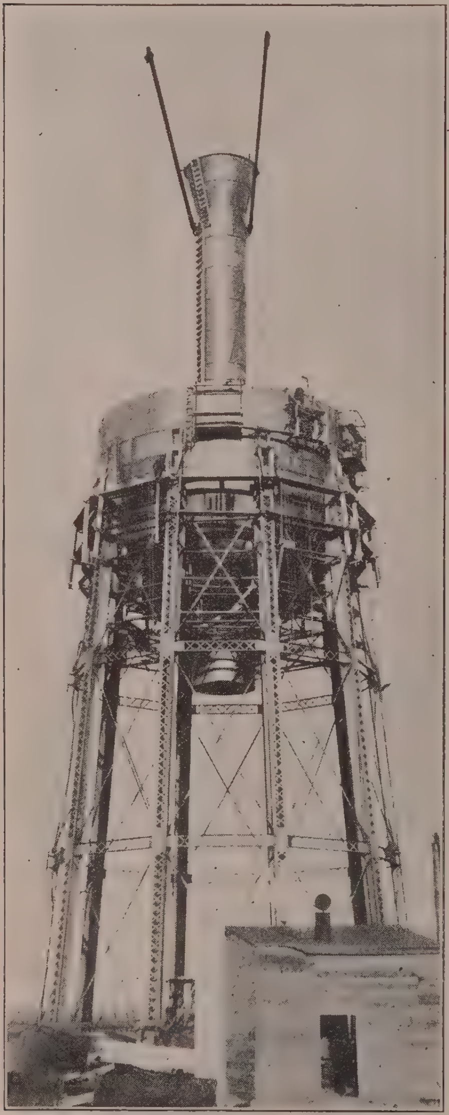
No new public water supply should be installed and no new source of water drawn from without the approval of the department.

Whenever in any case the requirements as to the operation of a water supply are in its opinion not met, the department should have the power to take over after due notice the operation of the system and continue such operation at the expense of the municipality.

Whenever the sewage-disposal plant of any institution on any watershed draining into a public water supply is operated in the opinion of the department in such a manner as to constitute a nuisance or a menace to public health, the department should have the power to take over the operation of such plant and to continue such operation as long as it may deem necessary for the protection of the public health, or until the abatement of the nuisance, the expense of such operation to be borne by the institution.

For the proper control of the water purification plants in the State, regular laboratory analyses of the treated water should be made by the municipality or water company owning the same, the results of which analyses should be promptly transmitted, together with all essential operating data, to the State Department of Health.

It should be the policy of the State Department of Health to make regular routine sanitary inspections and laboratory analyses of all public water supplies within the State.



TANK IN COURSE OF CONSTRUCTION

Salmon River Surge Tank

Design and Construction of the Tallest Standpipe Yet Built, Employing the Differential Principle to Minimize Surges

A SURGE tank 185 ft. high above the top of its foundation and operating on the differential principle has been erected at the hydroelectric plant of the Salmon River Power Company, near Altmar, N. Y. This surge tank was briefly mentioned in the description of the development in the Engineering Record of June 13, page 671. Its function is to absorb the energy of a column of water 9625 ft. long and from 11 to 12 ft. in diameter. As a plain cylindrical surge tank would have been prohibitive for several reasons, such as thickness of metal in its lower part, height required to prevent spilling, etc., and as the differential principle lends itself admirably to the absorption of energy, it was decided to build this type.

The standpipe, consequently, consists of an elevated steel tank, 50 ft. in diameter and 80 ft. high, connected with the distributor for the power-house penstocks by a 12-ft. riser. Inside of the tank is an interior riser 10 ft. in diameter, flaring to a diameter of 15 ft. and 10 ft. 8 in. at the top and bottom respectively. As the riser to the tank is 12 ft. in diameter there is an annular opening 8 in. wide between the risers at the bottom of the tank. This opening is divided into twelve spaces forming the "differential ports." The function of the surge tank, therefore, is as follows: When a certain part of the load is thrown off the station and the upward wave begins in the riser of the tank, a part of the volume of water is, through these ports, deflected into the main tank, reducing the ultimate height of the level in the riser. When the wave recedes, water will flow through the ports into the riser, thus decreasing the depth of the surge. The greater the difference in elevation between the levels in the tank and the riser the larger the quantity of flow through the ports. The action, therefore, is to throttle the surges. On the accompanying diagram is shown a typical case comparing the surges in the riser of a differential surge tank with the surges that would have taken place had the interior riser been eliminated from the design.

Features in the design of the 850-ton structure include the concrete ring foundation, the simple connection of tower to columns and the method of eliminating the usual outside balcony by an internal system of horizontal struts at the top of the bowl bottom.

Foundation Ring

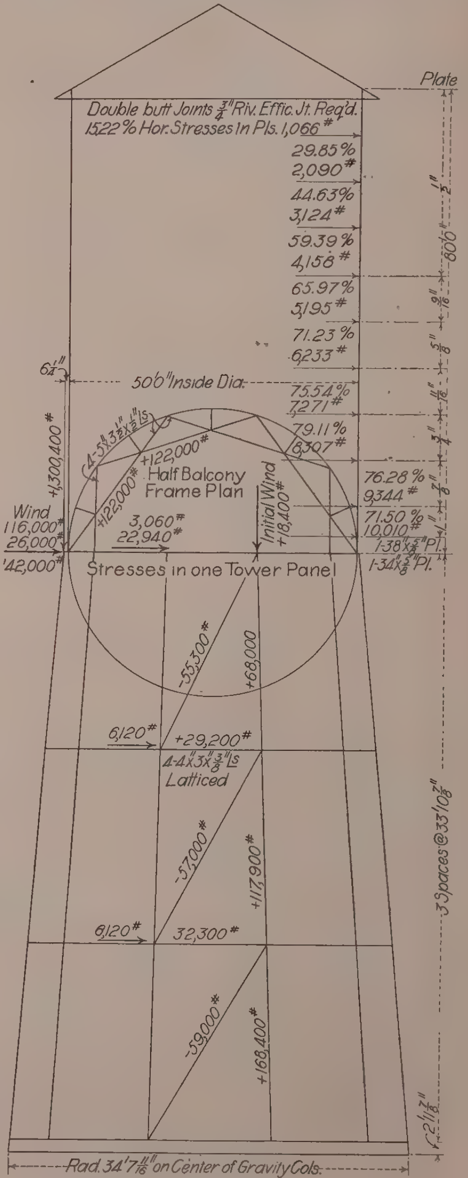
While each of the ten columns of the tower rests on a separate pedestal, the bottom portion, about 8 ft. below the natural surface of glacial drift, composed of clay and boulders, is connected to the adjoining pedestal by a concrete slab 4 ft. thick and 14 ft. wide. Reinforcing bars under the pedestal extend through the connection, forming a complete concrete ring.

Embedded in each pedestal is a double tier of steel I-beams forming a grillage to reduce the 756-ton load to 300 lb. per square inch on the concrete. Two 2½-in. bolts anchor the faced bed plate of the column to the five 24-in. I-beams in the top tier of the grillage. This grillage is set at an angle in the concrete so that the column thrust is normal, thus simplifying the details of the column footing and placing the tendency to spread in the concrete.

Interest in the massive columns, which are made up of two 8 x 8 x ¾-in. angles, two 6 x 6 x 1-in. angles, two 24 x 15/16-in. plates and one 34 x ¾-in. plate, all with an area of 115 sq. in. and a radius of gyration of 9.3 in., centers on the tank connection. Here 650 tons must be transmitted to each column. It is done by eight rows of rivets, thirty-two in each row, through the 34-in. plate and angles on the inner side of the column. Beginning 14 ft. 3 in. from the top, the column is reduced in thickness from 25 in. to 15¼ in., the outer face being a straight line.

Internal Balcony

Joining every other one of the above connections, on the inside of the tank, are interlocked struts to care for the horizontal thrust of the legs, due to their inclination, plus the wind load. These struts are Howe trusses made up of four 5 x 3½ x ½-in. angles latticed together. They form a balcony inside instead of the usual external balcony. They are interconnected at the



DESIGN OF DEAD AND WIND LOADS

third points, and for lateral stiffness of the bowl a strut connects these third points with the annular ring of angles inside of the shell. Further stiffening of the bowl is attained by two horizontal ⅝-in. bands in the shape of butt straps, 34 and 38 in. wide. A factor of safety of five was used throughout. The hemispherical bottom is designed to be self-supporting under full load with the aid of the ⅝-in. bands and balcony braces. The main stresses in the bottom are from the water, the 10-ft. riser and the 12-ft. opening for the riser connection.

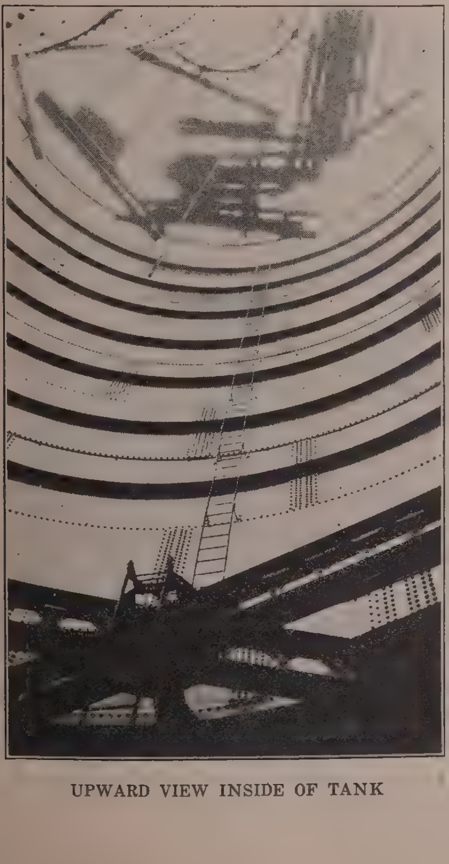
Construction of the outer 50-ft. tank the full 205-ft. height was at first considered, but was very soon abandoned, as with the factor of safety of five the calculated plate thickness reached the impracticable figure of nearly 3 in.

RISER PIPE AND EXTENSION

Above the bottom of the tank the 100-ft. extension of the riser is 10 ft. in diameter. It is centered by twelve I-beams inside of that portion of the lower 12-ft. riser above the expansion joint, which is located 8 ft. below the connection with the bowl. The annular spaces between the I-beams constitute the "differential ports" which damp the total inflow and outflow. These ports can be plugged with wooden blocks, so as to allow adjustment in area, if found necessary.

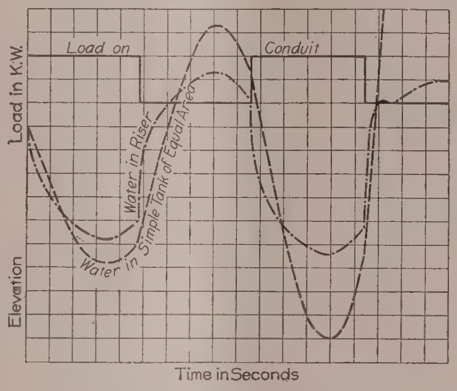
Due to the retarded outflow, external stresses, calculated in the same way as those due to internal hydrostatic loading, but opposite in direction, are set up in the inner pipe. Reinforcement to care for these stresses under the 65-ft. head, which tend to cave the pipe inward, consists of 4 x 4 x ½-in. angles every 3½ ft. vertically. Similarly the main tank was reinforced with angle rings to withstand the effect of a partial vacuum estimated at 100 lb. per square foot, although swinging doors were placed in the ceiling of the covering for the air to get in and out.

A special boom derrick, used in erection and believed to be the tallest yet constructed, was designed for this work by the Clyde Iron Works and fabricated by the contractor. It had a 140-ft. mast, 125-ft. boom and a crane capacity of 15 tons. Seven ⅞-in. steel cables, one 1500 ft. long, anchored it in place.

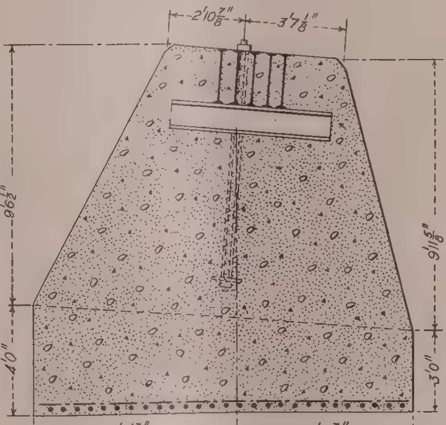

UPWARD VIEW INSIDE OF TANK

First in order of construction, following the setting of the derrick at the center point, was the erection of the columns in three sections of about 10 tons each. Then to the first course of the main tank were hung the twenty curved pieces of the bowl, all of which were bolted up before starting to rivet. The bowl had been bolted up in the shop in an inverted position and all holes reamed. The ⅞-in. plates were pressed cold by a hydraulic press. Trial tests indicated considerable spring in the plates and the dies were turned down to obtain the correct radius.

After the bowl bottom and the section supporting the inner upper riser pipes were riveted the boom of the derrick was removed. The eight sections of the inner pipe were riveted together around the mast and later raised and riveted in place. The remaining sections were added to the top by ordinary methods of constructing a steel smokestack. Two 40-ft. swinging booms were attached to the center pipe to hoist the shell plates and roof material. The down leg was built from the bottom of the bowl downward, each course being hung on in succession. Previous to placing the last ring the mast was removed in sections.

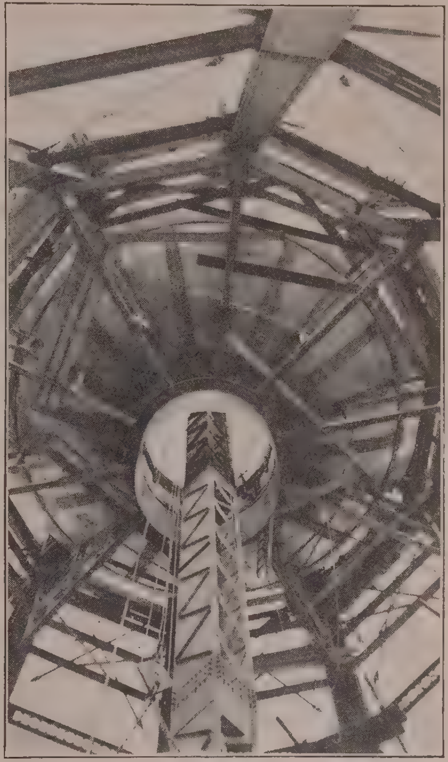


COMPARISON OF SURGES IN TANK



DETAIL OF COLUMN FOUNDATION BLOCK

Eyes on the sway rods, made of 4 x 1¼-in. bars, in the panels between the columns were formed in a hydraulic press, only one weld being made. The portion at the center, upset to 3⅛ in. in diameter for the turnbuckle, was also formed cold in a hydraulic press. Tests under the maximum load, 29½ tons, applied at the Pittsburgh Testing Laboratory, failed to show any elongation by the extensometers used.


UPWARD VIEW BELOW TANK

The Kennicott Company, of Chicago Heights, Ill., designed, fabricated and erected the tank as subcontractor to the Erie Construction Company, general contractor. R. D. Johnson, hydraulic engineer of the power company and inventor of this type of tank, drew up the specifications and was constantly in consultation with the Kennicott Company on the design. All final plans were approved by V. G. Converse, chief engineer of the Salmon River Power Company.

Milwaukee Refuse Collection and Disposal for 1913

ALL complaints of inadequate or unsatisfactory service in the collection of garbage in Milwaukee are given attention by dispatching a special collection wagon to the scene of the complaint. An average of 106 collectors returned 63,786 loads, or 40,752 tons, at a cost of $2.42 per ton, according to the 1913 report of the Department of Public Works. Segregated, the refuse consisted of 77.44 per cent of garbage, 4.32 per cent rubbish, 17.19 per cent ashes, and 1.05 per cent manure. Per ton of total refuse the labor cost was $1.07 per ton; for garbage alone it was $1.38 per ton.

For ashes and rubbish collection the wagon capacity was increased from 2½ to 3 tons, resulting in an increase of from 26,202 cu. yd. to 319,848 cu. yd. for the year and at a reduction in cost over 1912 from $229,344 to $225,421. The monthly yardage varies from 8133 in July to 46,578 in January. Four per cent only went to the incinerator, the remainder being hauled to dumps.

About seventy men working in three shifts are required for proper operation of the plant, which has four 75-ton units, and was placed in operation in 1910 at a cost of $212,000. A description was given in the Engineering Record of July 16, 1910, page 60.

Generally sufficient combustible material is present to incinerate all refuse material properly, except for short periods in the summer months. The fuel added is coke breeze obtained free, the city paying only for the freight. A power plant has been installed to convert the steam into electricity furnished to the pumping stations. The revenue credited to the incinerator will lower the cost of burning about 20 cents per ton.

Book Reviews

Farmers' Bulletin 597 of the U. S. Department of Agriculture discusses under the title of "The Road Drag and How It Is Used" the best methods of maintaining earth roads in good condition with the device. The bulletin describes the split-log drag, explains its use and tells the best time to use it.

"The Wet Lands of Southern Louisiana and Their Drainage" is the title of Bulletin 71 of the U. S. Department of Agriculture. Its author is Charles W. Okey, drainage engineer. Attention is called to the fact that about 35 per cent of the total area of Louisiana is classed as swamp and overflowed land, making the drainage of those territories a public improvement of great importance to the future wealth and prosperity of the State. (Washington, D. C., U. S. Department of Agriculture.)

Water-Supply Paper 323, recently issued by the U. S. Geological Survey, is an addition to the literature dealing with the surface water supply of the United States. This volume, which is "Part III, Ohio River Basin," is written by A. H. Horton, W. E. Hall and H. J. Jackson, and contains 118 pages. The results presented are of measurements of flow made on streams in the United States during 1912. Water-Supply Paper 340B, "Stream-Gaging Stations and Publications Relating to Water Resources 1885-1913, Part II, South Atlantic Coast and Eastern Gulf of Mexico Drainage Basins," has also been published recently. The pamphlet is written by D. B. Wood. (Washington, D. C., U. S. Geological Survey.)

"Acoustics of Auditoriums," by F. R. Watson, has just been issued as Bulletin 73 of the Engineering Experiment Station of the University of Illinois. The faulty acoustics of many auditoriums is called to the attention of the public at frequent intervals, either in the newspapers, where an account is given of some faulty structure, or by attendance in an auditorium where the echoes and reverberations of sound make it difficult to hear. The auditorium at the University of Illinois presented such a case. A veritable chaos of sound was set up and auditors all over the house experienced difficulty in understanding what was said on the stage. An investigation of the acoustical properties of the auditorium was begun in 1908 and continued for six years. Echoes were located by using an alternating-current arc light at the focus of a parabolic reflector. In addition to the light, the arc gave forth a hissing sound which was of short wave length and therefore experienced but little diffraction. The bundle of light rays sent out by the arc light was, therefore, accompanied by a bundle of sound, both coming from the same source and subject to the same law of reflection. The path of the sound was found by noting the position of the spot of light on the wall, and, by using small mirrors, the echoes were traced by the reflected light and confirmed by means of the sound. Curtains and canvases were hung in various positions in the auditorium so as to stop or deaden these reverberations, with the result that the acoustics were greatly improved. The bulletin is illustrated with numerous photographs showing the methods used to improve the acoustics and with a number of cuts showing the character and paths of the echoes in the auditorium. Copies may be obtained gratis upon application to C. R. Richards, acting director of the Engineering Experiment Station, University of Illinois, Urbana, Ill.

Reinforced Concrete Railway Structures. By J. D. W. Ball, Assoc. M. Inst. C. E. Cloth, 5½ x 8½ in.; 213 pages; 123 illustrations. New York, D. Van Nostrand Company, $2.50 net.

(Reviewed by F. H. Constant, Professor of Structural Engineering, University of Minnesota, Minneapolis.)

This is an English work on a practical subject. Its scope is indicated by the chapter headings as follows: Preliminary considerations, bending stresses, shearing stresses, floors and buildings, foundations and rafts, retaining walls, bridges, arched bridges, sleepers, fence posts, etc., and summary of notation. The American reader will feel more at home in the early part of the book in which the theoretical treatment of beams is not materially different from that given in elementary works in this country. Exception may be taken to the method of designing stirrups for shearing stresses in the steel instead of diagonal tension in the concrete. The treatment of columns is wholly inadequate, especially in regard to the effect of hooping. A few diagrams showing the graphic curve solutions of formulæ are interspersed in the text and, as is usual, are useful in expeditious computation.

The greater part of the book is concerned with the practical design of various types of railway structures, such as spread foundations and column footings, cantilever and counterforted retaining walls, girder and arch bridges, etc. It is largely descriptive and illustrated by text cuts and numerical examples worked out in detail. There is very little statement of rigorous analysis, the author depending almost wholly upon the numerical problems for the exposition of the necessary theory. Many approximate processes are used which will not strike the informed reader as justifiable or accurate. The treatment of the arch is especially inadequate.

The book will not interest American designers, not only because types and methods are different, but especially because the field is covered in a much better way in our own literature.

Letters to the Editor

Rapid Stadia Work

SIR: The photographs and description of rapid transit work of our field parties published in the Engineering Record of June 6, 1914, page 639, were furnished by one of our field men without the knowledge of Mr. Hidinger [vice-president of the company] or the writer. We have looked up our records to see whether the statements were correct. The best record we have been able to locate under conditions outlined in this statement is of 585 side shots in an 8-hr. day, the instrumentman doing his own recording.

We also find the following record, which is interesting. In one working day, stated by the transitman to be a little less than 8 hr., a party consisting of a transitman, recorder and three rodmen, did the following work:

Topography was taken along 3¾ mi. of base line which had already been located. The transit was set up over seventeen stations, in each case being oriented by observation on the last station, elevations being carried forward at the same time. During this day's work 1023 side shots were recorded, together with eight sketches and forty notations of topographic details. Each side shot was recorded by noting the horizontal angle as read from the vernier, the distance read by stadia, and the elevation by vertical angles. The notes mentioned in the foregoing occupied forty-four double pages of standard notebook. The work was in hilly land mostly cleared, the average distance between the points being about 200 ft.

This record is part of the usual work of the party and is not a special test to determine speed, though it is probably the best day's work we have recorded. Other records show between 900 and 1000 side shots, using three rodmen and a recorder, and about 750 side shots using two rodmen and a recorder. The recorder is a rodman assigned to this work.

Morgan Engineering Company,
By Arthur E. Morgan.
Memphis, Tenn.

Design of Reinforced-Concrete Retaining Walls

SIR: It seems to the writer that C. W. Martin, in his remarks in your issue of June 6, page 657, on a letter on retaining walls by D. M. Becker, in the issue of Jan. 17, page 86, has gone rather wide of the mark and has raised a storm of formulæ in a teacup problem.

Mr. Martin's analysis of a member under bending and direct stresses rests on a fallacy expressed in his equation (a)

$$N - f_c bkd/2 + f_s A = O$$

where N = axial load; f_c, allowable compressive unit stress; f_s, allowable tensile unit stress; b, width of member; d, depth of member; A, area of reinforcement, and kd distance of neutral axis from face in compression.

Such an equation is faulty for several reasons. In the first place, both N and $f_c bkd/2$ are of the same kind and act in the same direction, and, therefore, they ought to be of the same sign, and if we wish to add the forces on one side of the section we would have $N + f_c bkd/2 - f_s A$ or $- f_c bkd/2 - N + f_s A$, according as we consider compression positive or negative.

In the second place, there is no scientific reason whatever which makes the sum of the forces equal to zero. All that is required for equilibrium is that the forces acting on one side of a section be equal and opposite to the forces acting on the other side, or that the sum of the moments of all the forces acting on one side of the section be equal to zero. These conditions are fulfilled in the case under consideration regardless of the relation between the axial load N and the forces resisting the bending moment.

In the case of a member under a simple bending moment it so happens that the forces on either side of a section add up to zero because they are a couple and, therefore, they are of the same magnitude but opposite direction. But it does not follow that if an axial load be introduced the sum of the forces on one side of the section will still be zero. As Mr. Martin's equations for the values of p' and R' are derived from his formula (a), which is wrong, therefore, his other equations are also wrong.

In reality the problem of a member under a bending moment and an axial stress is a very simple one. Let us take, for instance, an element of a slab forming one side of a box conduit. Here we have a beam under a bending moment due to the earth pressure and an axial load from the earth and other superimposed loads on the corresponding slab element forming the top of the box. On one face we have a unit compressive stress due to the bending moment and a unit compressive stress due to the axial load. The sum of the two will be the actual unit compressive stress on that face. On the other face we have a unit tensile stress due to the bending moment and a unit compressive stress due to the axial load. The algebraic sum of these two stresses will be the actual unit stress, which may be tensile or compressive.

This means that in designing the slab for the bending moment we have to reckon with a material—concrete—which will be weakened by the axial load, and another material—steel—which will be strengthened by the same axial load. Therefore, in making the computations, we must deduct the unit stress due to the axial load from the allowable unit compressive stress, and add the same unit stress to the allowable unit tensile stress.

In other words, let f_c = allowable unit compressive stress; f_s, allowable unit tensile stress, and h, unit stress due to axial load. Then

$$M = A(f_s + h)jd = bkdjd(f_c - h)/2$$

making

$$A = pbd = \frac{f_c - h}{2(f_s + h)} bd$$

and

$$k = \frac{n(f_c - h)}{n(f_c - h) + (f_s + h)}$$

where M = bending moment; A, area of reinforcement; jd, moment arm of section; kd, distance of neutral axis from compression face; d, distance of reinforcement from compression face; and n, ratio of moduli of materials.

Of course, if the axial force were tensile instead of compressive we would have

$$M = A(f_s - h)jd = bkdjd(f_c + h)/2$$

making

$$A = pbd = \frac{f_c + h}{2(f_s - h)} bd$$

and

$$k = \frac{n(f_c + h)}{n(f_c + h) + (f_s - h)}$$

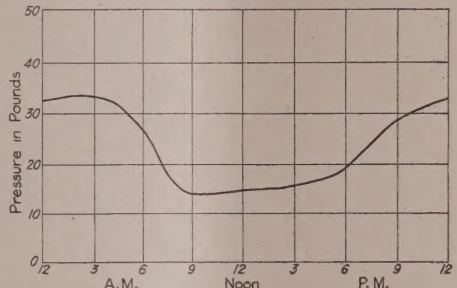
In the case of L or T-shaped retaining walls the axial load is so small as not to be worth considering, but in box conduits and culverts it may be quite large.

M. J. LORENTE,
Cambridge, Mass. Civil Engineer.

Relation between Pressure and Waste in Water Supply

SIR: One of the ideals sought by many municipal waterworks men is to maintain a "good pressure." If they can supply water at 80 lb. pressure it is considered much more commendable than if they only maintain a pressure of 40 lb. There is a tendency to ignore the fact that as the pressure increases all leakage and waste also increase, while the amount of water actually used remains practically the same.

At least 20 per cent. of the plumbing fixtures in a large city leak constantly, and there are always underground leaks which do not appear on the surface; consequently each pound of added pressure correspondingly increases the waste from these sources. Moreover, the higher the pressure, the more water will be used legitimately—that is, a person drawing a gallon from a spigot when the pressure is low is likely to draw two or more gallons when the pressure is high. The per-capita consumption is in this respect a very elastic quantity and increases or decreases according to the pressure maintained. The water actually used is only a small percentage of this so-called per-capita consumption.



PRESSURE THROUGHOUT 24-HOUR DAY

From the standpoint of economy the proper pressure to be maintained in any neighborhood or district is undoubtedly the lowest which will give a sufficient supply, and it is likely that in most towns a considerable amount of time and effort might be profitably spent in reducing and adjusting the pressures in the various districts according to their actual requirements. This minimum pressure depends on the character of the district or locality and is not difficult to determine. It should seldom exceed 40 lb., and in most residential districts 25 to 30 lb. is ample.

If the pressures taken on some fire hydrants in a residential district during twenty-four hours are plotted they will be found lowest from 8 a. m. till noon, then rising gradually until 7 or 8 p. m., and more rapidly from 8 p. m. to midnight, and are at a maximum from midnight to 5 a. m. The accompanying diagram shows the actual pressures recorded in such a district.

The fact that 15 lb. supplied this district without any complaints during the busiest portion of the day is fairly good evidence that there was no necessity for the pressure of 35 lb. at night. There are, however, some manufacturing districts where the nightly increase in pressure is depended on to fill roof tanks, though whether they are legally entitled to such service is questionable.

The experiment was recently tried of cutting down the night pressure in a certain district to a minimum, this minimum being assumed to be just above the point at which complaints began to come in. Men were stationed to watch pressure gages on fire hydrants at several convenient points throughout the district. Beginning at midnight the outlet control valves at the reservoir were closed down a few turns at a time, the men operating them being in touch by telephone with those at the various pressure gages throughout the district and with the night operator at city hall to whom complaints of poor pressure come in. The number of turns on the valves was counted, and the point finally found to which these valves could be closed each night without causing any actual shortage of water. In case of fire the proper valve was opened.

The first tangible result was a somewhat excited inquiry from the filter plant above wanting to know why the water was not being taken, followed by a reduction of the rates on all the filters, this being, of course, followed by a corresponding reduction at the river pumping station. The consumption in the district fell during the week from 39,000,000 to 34,000,000 gal. daily without any inconvenience to residents.

The cost of operating the valves at night is small. Such outlet valves from basins should, however, be fitted with motors or hydraulic cylinders so as to be operated easily by one man. Basins should properly be filled up at night from above, with the outlet valves partly closed, rather than by backing up from the city.

JOHN S. ELY,
Assistant Engineer, Bureau of Water.
Philadelphia.

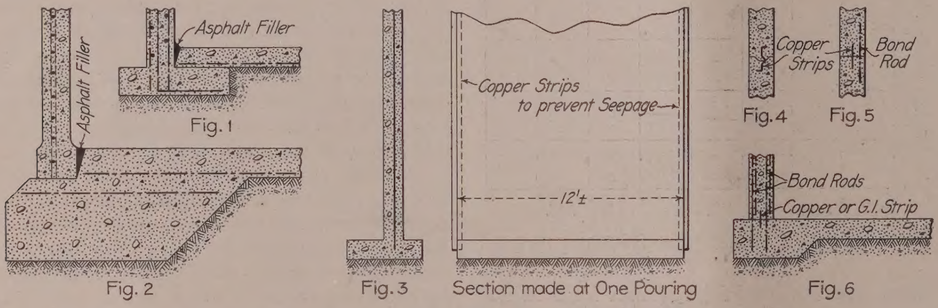
Reinforced-Concrete Standpipe with Sliding Base

SIR: The writer's attention has been called to a discussion in the Engineering Record of March 7, 1914, page 290, entitled "Reinforced-Concrete Standpipe with Sliding Base." Inasmuch as the parties in the discussion have referred to the writer's letter in the "Engineering News" of Sept. 7, 1911, and have, it would seem, inadvertently misconstrued the thought which that letter was written to develop, the writer wishes to explain his position more fully, for it is believed that more can be said that will be of interest to the readers of the Engineering Record.

The details of the junction of the walls with the base of the standpipe advocated by me in the letter referred to above are shown in Fig. 1. The following statements of Mr. Mueser in regard to the principles of the assumption involved could not be better said, and are quoted for this reason. "As stated in the article on the Fulton standpipe, the writer considers, as the main purpose in disconnecting the wall and bottom, the possibility of having the bottom serve as a foundation only, taking care of all vertical pressure resulting from the weight of the water and from the weight of the circular wall, and of having the wall take care of only the horizontal pressures, which it can do best when designed as a simple shell." Mr. Mueser goes on to say that the walls should rest on the base and that a slip joint should be provided of such a nature that the coefficient of friction between the wall and base shall be a minimum. Fig. 2 shows the manner in which Mr. Mueser contemplates the construction of the Sanderson standpipe.

It occurs to the writer that Mr. Mueser has carried the idea of a sliding base so far as to overdo the requirements. Observe Fig. 2. The pressure between the walls and the base of the standpipe is directly proportional to the height of the walls; it also depends on the pressure of the wind on the sides of the standpipe, but is independent of the amount of water in the standpipe.

It is conceded that if the joint filler remains always plastic the matter of the wind pressure named above would not enter into the problem. On the other hand, it would be presumptuous to assume that any joint filler made would remain continually plastic. We should need then a tight-fitting pressure connection between the side walls and the bottom, first, to decrease the amount of leakage in case the joint filler fails at some point, and, second, so made that the weight of the water will give additional stability to the standpipe.



DETAILS OF TANKS DISCONNECTED AT BASE

The need for this additional stability will depend on the height of the standpipe, thickness of the walls and possible wind pressure. When the standpipe is 100 ft. or more in height and 20 or 25 ft. in diameter, the effect of a 90 or 100-mi. wind will be to reduce the pressure on the windward side to practically nothing; if, then, the standpipe is nearly full of water a surge would result and the joint would open sufficiently to produce a leak.

It was my desire to make the joint between the base and the walls in such a way that water on the base slab would increase the pressure between the joining surfaces as the head increases and make that pressure independent of any external forces. I first used the type of footing and wall shown in Fig. 1 in the spring of 1911 in a reservoir built at Fort Terry, N. Y. The depth was approximately 12 ft., the reservoir being 37 x 72 ft., rectangular in section with rounded corners on a radius of approximately 5 ft. There is a reinforced-concrete roof on the reservoir, supported by columns resting on the floor, which was 12 in. thick, reinforced with a wire mesh. The contractor desired to do the work by hand mixing with a small gang, five or six men. In order to make it practicable to do this work with a gang of the above size, the walls were cast in six parts or in lengths of approximately 12 ft., one length being cast at each pour. In the vertical joints a strip of copper was used. The first one or two were bent and put in as shown in Fig. 4. The V at the center of the strip was made to allow for expansion or contraction of the wall. The right-angle bends were to anchor the strip of copper to the walls. The last few strips were made 4½ in. wide and put in as shown in Fig. 5. These joints were entirely tight when the reservoir was tested. There were, however, two leaks in the bodies of two different units, caused by the fact that the concrete had not been kept level and the cement had been allowed to run out, leaving the aggregate improperly proportioned. The leaks were easily repaired by scraping off the outside skin and plastering the wall with mortar.

After the reservoir was completed the surface of the walls exposed to the action of the water was washed over with about a 1:1 cement wash and rubbed with a cloth float. This process filled all air holes. Three applications of Sylvester's solution were made. As far as could be determined, the walls were absolutely non-absorbent.

For reservoirs, standpipes or cisterns where the head is less than 50 ft., the method of construction shown in Fig. 6 is recommended, so far as the junction of the walls and floor is concerned.

In 1912 thirty-four reinforced-concrete cisterns were constructed at Fort H. G. Wright, the greater part under the direction of the writer. The diameters of the cisterns varied between 8 and 16 ft. and the average depth for water was 12 ft. The cisterns were all below the ground surface and two of them had their bottoms 4 ft. below the ground-water level. The excavations were made to the required elevation for the bottom of the cisterns and at a single pour the concrete was put in over the entire area of the base plus the side walls, and for a projection outside of the walls of 6 or more inches. Before the concrete base had set a 4-in. strip of galvanized iron was then pressed into it so as to project 2 in. into the base and stand 2 in. above the base to engage the walls when they were poured. This made a union of base to side walls practically as shown in Fig. 6. Bond rods were put into the base and allowed to project above the same to engage the walls also.

Five or six of the first cisterns were carefully tested with water; no leaks developed at the junction of the base and walls, nor at any part of the wall surfaces. The cisterns were of such size as would be well adapted for construction in two pours—the base for the first pour and the side walls for the second pour. The galvanized iron served to cut off possible seepage between the two pours.

The difficulty in producing waterproof concrete structures is more generally due to lack of necessary engineering consideration than to lack of proper workmanship in construction. The assumed unit stress in the steel should be so taken that it will not be necessary for the concrete to stretch beyond the elastic limit before the steel develops the stress required. This value will generally be between 7000 and 10,000 lb. per square inch, or, roughly, about one-half of what is used in most building work. All expansion joints or sections between pours should be provided with the necessary metallic cutoff strips.

The use of waterproofing compounds other than some of the more simple, like Sylvester's solution, or a wash of hydrated lime and cement mortar, will be unnecessary. The three watch words for the designer should be comparatively low unit stresses in the steel, ample expansion and shrinkage joints with metallic cutoffs, and a definite decision as to the manner of executing the work of construction.

G. L. BILDERBECK,
Assistant Engineer, Rivers, Harbors and Bridges Commission, State of Connecticut.
New London, Conn.

Comparisons of Systems of Flood Control

SIR: The timely and valuable communications by H. A. Petterson, concluded in your number of May 16, on the subject of "Comparisons of Systems of Flood Control," deserve the earnest consideration of every citizen interested in the welfare of this country, and I hope you will encourage other contributions on the subject of flood control, since it involves one of the most important features of conservation, and legislation must be shaped to meet the requirements of our ever-advancing civilization.

The engineers of this country should lend themselves freely to their duty, for it is clearly their duty to lead in this matter, which can make no substantial progress until a common policy is adopted, or at least recognized, so that our energies as a Nation may be directed effectively.

W. R. SMITH, JR.,
Wm. Russel Smith Company.
York, Pa.

Confusion in Paving Specifications

SIR: I notice in an editorial entitled "Confusion in Paving Specifications" which appears in the Engineering Record of June 13, 1914, the statement that the method of test recommended by the Association for Standardizing Paving Specifications, adopted at its meeting in Pittsburgh in February, 1913, states that the sample to be tested should have a weight of 50 grams. This statement is incorrect.

It is clearly shown in the printed fourth annual report of the association, on page 71, that the volatilization test should be made with 20 grams of bituminous material.

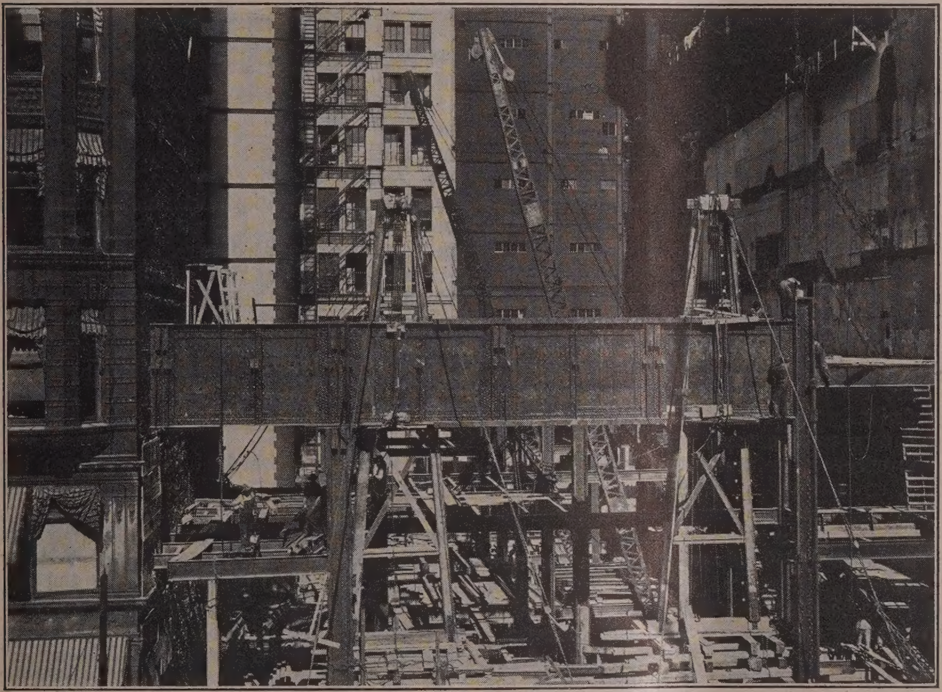
OTTO H. KLEIN,
Vice-Chairman of Asphalt Committee.
New York City.

GARBAGE COLLECTION COSTS in Chicago during 1913 from thirty-three wards were $3.69 per ton for the 109,809 tons removed. More than 1,500,000 cu. yd. of ashes in 280,000 loads were collected and hauled to dumps at 67 cents per cubic yard. Street and alley cleaning necessitated the removal of 816,870 cu. yd., or 216,578 loads, from 3,076,968,808 sq. yd. of surface at a cost of 40.8 cents per 1000 sq. yd. Snow removal in the downtown ward cost 38.6 cents per cubic yard for 132,230 cu. yd. in 32,326 loads. In 1912 garbage removal cost $3.90; ashes and refuse, 63.9 cents per cubic yard, and street and alley cleaning, 37.7 cents per 1000 sq. yd.

Cantilever Store-Front Girder

Twenty-two-Horse Team Hauls Heavy Girder Over Downtown Streets in Chicago

A HEAVY girder, 9 ft. deep, with a cantilever end overhanging 15½ ft., supports the northwest corner row of columns in the new 19-story Stevens Building on State Street, Chicago. The 92 x 150-ft. lot on which the present structure is being erected is just south of the Columbus Memorial Building, which rests on a floating foundation. It was not considered advisable to sink a caisson for the corner columns, so the deep girder was used at the third floor level, the second floor being suspended from the end and two intermediate panel points, thus giving a maximum of show-window space on the two lower floors. The girder is 59 ft. 2¾ in. long and the supporting columns rest on one 10½-ft. caisson and one 8½-ft. caisson excavated to rock. At the heavy section it consists of eight chair angles, 8 x 8 x 1 in., four side angles, 8 x 6 x ⅞ in., four side plates, 15 x ⅞ in., six cover plates, 20 x 13/16 in., and two web plates, 108 x 11/16 in. More than 24,000 holes were punched for the 1-in. rivets.



LARGE CANTILEVER GIRDER FOR STORE FRONT AT CHICAGO

TRANSPORTATION

After fabrication in the shops at Gary the girder was shipped upright on two flat cars over the Michigan Central to the Randolph Street station. Here a wrecking locomotive crane was borrowed from the Illinois Central to unload the girder, placing it flat on the ground. The chains attached to the three panel points were released and secured to the sides to raise it so as to permit backing the truck under the girder. Unloading from the car started at 7:30 a.m. June 14. By 9:30 a.m. the wagon was loaded and by 11 a.m. the girder stood in front of the building, five blocks of granite block and two of asphalt laid in 1899 having been traversed without more serious damage than two cracked manhole covers and a few crossing stones. Permit could only be obtained from the city officials for moving the girder on Sunday, on account of the congestion of traffic on State Street. Unloading consisted of jacking up the girder, inserting two 10 x 16-in. skids, 50 ft. long, each carrying two greased rails, and hauling the girder by electric hoists over the sidewalk to the foot of the columns which were to form its support. This took until 2:40 p.m. Street cars in the interval were diverted to Dearborn Street and Wabash Avenue. The skids were cribbed practically the whole length and at the same elevation as the top of the wagon.

Hauling was done by the Pennoyer-Merchants Transfer Company with twenty-two dray horses, nearly all weighing 1 ton each. The reach wagon weighs 5 tons and has a capacity of 75 tons. The wheels are of solid oak, 64 in. in diameter and 7 in. in thickness. The bolsters are of 12 x 14-in. oak and the reach is 8 in. square.

For erection two A-frames, 44 ft. in height, were constructed of 12 x 12-in. legs, cap and base. No load from cribbing was permissible near the north end, so the head block on one of the frames was strengthened by the addition of two 12-in. channels and placed 20 ft. south of the cantilever end. The other A-frame was placed 7 ft. from the opposite end. The spread of the legs at the head block was 2 ft. 1 in. and at the base 10 ft. All four joints were reinforced with ½ x 6-in. straps.

RAISING THE GIRDER

To raise the girder the heavy load was carried by five strands of 1-in. cable and the other end by a chain and two strands of ¾-in. cable. Leads from the four-sheave blocks went to a 40-hp and a 100-hp Thomas electric hoist in the basement. The A-frames were inclined enough to bring the center of the head blocks over the column centers, so that the girder skidded up the side of the columns and swung over into place. Just 2 min. less than 1 hr. was required for the raising, which was done June 16. In place there was a 9-ft. space between the top of the girder and the head blocks.

The George A. Fuller Company, with W. A. Taylor as superintendent, is the contractor for the owners, Charles A. Stevens & Brothers. D. H. Burnham & Company are the architects and designed the girder. Purdy & Henderson made the shop drawings for the American Bridge Company, which fabricated the steel.

Rapid Installation of a Temporary Booster Pump in San Diego

Rush Methods Used in Manufacture of Pump to Avert a Water Famine

INCREASING the delivery of water through 15 miles of 24-in. wood-stave pipe from a 7,250,000-gal. gravity flow to 12,250,000 gal. daily by means of a booster pump was accomplished last season at San Diego, thus saving the city a water shortage. A description of the rush methods used to install the temporary plant to carry the city over an impending water famine until the permanent installation at Chollas Heights, at the upper end of the pipe line, could be put into running order appeared in the Engineering Record of Aug. 16, 1913, page 169. The following notes are taken from the annual report of the water department and indicate rush work also on the permanent plant.

Inasmuch as a 580-speed motor was the only one procurable under the contract-time limit, the type of pump selected for this service was of the duplex two-stage design, or one in which the intake was divided through two low-pressure cases mounted on either side of and discharging into a common high-pressure discharge case. This design combined all the advantages of a double-suction, single-runner pump with that of the two-stage turbine type and, at the same time, by dividing the total head of 90 ft. into two equal services, permitted the use of 24-in. impellers, thus reducing the impeller losses to a minimum.

DIMENSIONS BY TELEPHONE

Having determined the type of pump to be supplied, preliminary drawings were submitted, upon which, with a few minor changes, the contract was let, all details of general construction being drawn up as the work of manufacture progressed. As soon as the drawings commenced to take shape, dimensions were telephoned to the pattern-making department, where the work on core boxes and patterns was being rushed, and some of these were finished before the blueprints were dry. The work on the core boxes of the larger pieces preceded that on their respective patterns a sufficient space of time to allow the drying of the cores while the foundry was preparing the molds.

On receipt of the castings the machine department rushed their finishing by working three eight-hour shifts per day. The time of making the drawings, patterns, castings, and assembling and testing the pump occupied twenty-six days of the time allowed in the contract, the pump arriving at the dock in San Diego four days later.

The United Iron Works, of Oakland, Cal., which built and designed this pump, sent their representative to look after the transportation and installation of the machinery. The pump and motor were hauled over 10 miles of sage brush and cactus to the pumping site and were installed upon the concrete foundation which had been prepared by the water department. The teaming required ten horses on each wagon two days to make the trip over a mere track of a road, which necessitated planking over the soft places and blocking up under culverts. In some places the road was abandoned altogether and a new trail broken through the sage brush. Upon the arrival of the machinery at the pumping site the builders joined forces with the water department and the power company to complete the installation, which was finished on the afternoon of the last day of the contract-time allowance and was turned over in running order, without an accident of any kind during the course of construction.

SIZE OF PUMP

The pump when completed weighed 22,500 lb., was 21 ft. over all in length and required a truck with a width of 9-ft. wheel centers to haul the bed plate. The shaft, which extends through water-sealed packing boxes on both sides of the pump casings, is supported by two-ring oiling bearings, in addition to the motor shaft, by means of a flexible leather link coupling having four leather links with a combined cross-section of 48 sq. in. of leather.

When lowered to the foundation the twelve bolts set into the concrete by the water department entered the 1/8-in. clearance holes in the bed plate without marring a thread. The pump operated with an entire absence of vibration, and the pressures established by its operation along the entire length of the 15-mile pipe line on which it is installed came within 1½ ft. of that calculated. There has been no trouble whatsoever since the installation of this pump, although the water consumption reached as high as 12,500,000 gal. on Sept. 17.

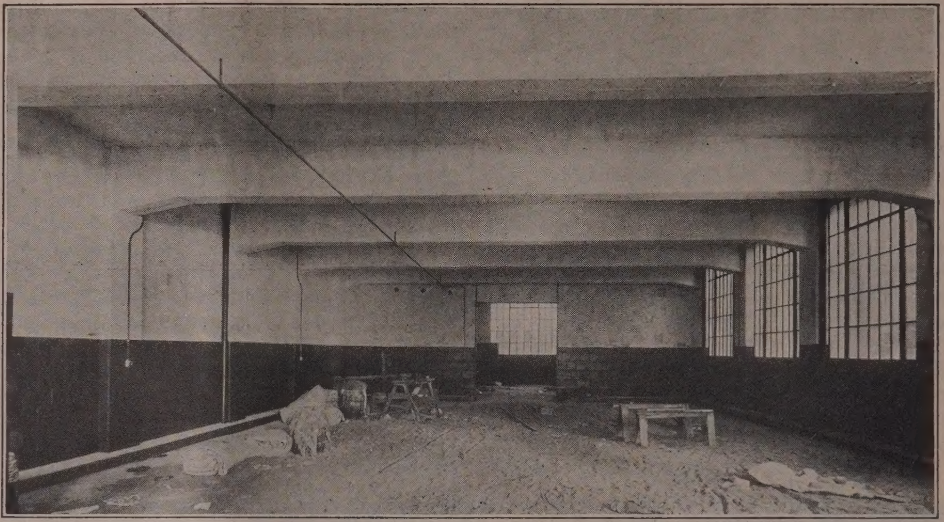
Major Herbert R. Fay, member of the common council, is superintendent of the San Diego Water Department and H. A. Whitney is hydraulic engineer and assistant superintendent.

Garage of Reinforced Concrete

One-Story Building in Which Columns and Girders Have Been Considered to Act as Portals

A GARAGE about 43 x 120 ft. in plan, without interior columns except around elevators, has recently been completed at Charlestown, Mass. At present it is a one-story building, but the foundations and columns are designed for four stories, and the roof will act as a future floor. The floor loads on all but the second floor are 125 lb. live. The second floor is designed for 200 lb. live load on the slab and 125 lb. on the beams. The latter are spaced 12 ft. on centers and are designed as T-beams. The working stresses and formulæ are those given in the last report of the Joint Committee. Although the future stories will not be built for several years, it was not thought advisable to increase the unit stresses in the concrete on this account.

Although the beams are only a single span in length, they were not designed as simple beams, as they are more or less restrained at the ends by the columns, and in turn produce bending in the columns. They were therefore designed in accordance with an article published in the Engineering Record of Oct. 15, 1910, entitled "Bending Moments in Concrete Wall Columns and Fixed Beams," in which the bending moments in beams and columns are deduced by taking account of the relative moments of inertia and lengths of each.



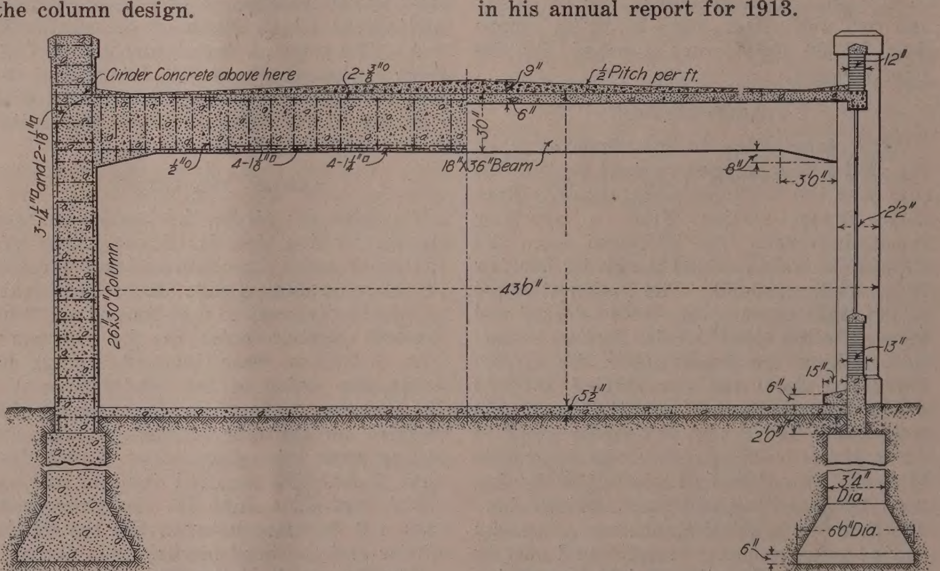
GARAGE FOR WHICH BEAMS ARE DESIGNED AS PORTALS

In this case for the four-story building the bending moment at the center of the beam became $wl^2/12$ and at the end of the beam $wl^2/16$. This negative moment of $wl^2/16$ was considered to cause a bending moment in the column below the floor of $wl^2/32$, and in the column above the floor of a like amount. In order to take care of the stresses in the future column above the floor, its reinforcing rods were extended sufficiently to develop the stresses in them and then were greased above the floor line. The concrete around them was made porous, so that when future stories are built it will be an easy matter to cut these columns off at the floor line, expose the reinforcement and lap it with that in the new columns. For the present one-story building the bending moments, due to a smaller live load, are less than in the four-story design. The negative moment, however, is transmitted only to the column below the beam and works out to be $wl^2/24$ in both the beam and the column. Therefore, the column is designed for this moment, combined with the direct load under these conditions, as well as for the bending moment and direct load under the four-story scheme. These computations show that the one-story scheme is the determining factor in the column design.

Another feature is the use of vertical stirrups without truss rods, a design that has been used successfully by the designers for the last five or six years. The slab is reinforced with expanded metal and the beams and columns are reinforced with mild steel corrugated bars.

The garage has been built for H. P. Hood & Sons, of Charlestown, Mass., and the concrete design was made by the contractor, the New England Concrete Construction Company, and checked by J. R. Worcester & Company.

THE SUPPLY MAIN leading from the terminus of the Weston aqueduct of the metropolitan water district, Boston, toward Chestnut Hill reservoir, which was laid in 1902 and 1903, has been cleaned for a distance of 6800 ft. at a cost of $384.25, or $0.056 per foot. The tubercles were removed from the interior of the pipe by laborers using wood and iron scrapers, and the pipe was then washed by the use of a spraying machine, using a jet from a ¼-in. diameter nozzle under a pressure of 200 lb. per square inch. The lighter material was washed out of the blowoffs, and 45 cu. ft. of the heavier material, composed principally of tubercles, were removed in pails. The work is described by Dexter Brackett, chief engineer of the Metropolitan Water Board, in his annual report for 1913.



Half Section Through Beam and Column Half Section Between Columns

CROSS-SECTION OF REINFORCED-CONCRETE GARAGE, SHOWING FRAMING OF BENTS